INDIAN WATER WORKS PRACTICE

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A PRACTICAL GUIDE FOR THE STUDENTS AND ENGINEERS INTERESTED AND ENGAGED IN THE DESIGN, CONSTRUCTION AND MANAGEMENT OF WATER WORKS IN THE TROPICS.

BY

K. C. BANERJEE

Executive Engineer, Public Health Department, Bengal.

Member of the Institution of Engineers (India); Member of the American Water Works Association; Associate Member of the American Public Health Association; Member of the Association of Engineers; Visiting Lecturer, Bengal Engineering College, Examiner in Sanitary Engineering, University of Calcutta.

WITH AN INTRODUCTION BY

G. BRANSBY WILLIAMS,

M. INST. C. E., M. I. MECH. E., M. I. E. (IND.), ETC.

Late Chief Engineer, Public Health Department, Bengal.

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To

GEORGE BRANSBY WILLIAMS, Esq.,

M. Inst. C.E., M.I.E. (Ind.),

LATE CHIEF ENGINEER,

PUBLIC HEALTH DEPARTMENT, BENGAL.

In Grateful Remembrance of the

Great Service Rendered by him to the Cause of

SANITATION IN BENGAL

AND OF THE

Patronage and Inspiration received during the

Author's Association with him.

INTRODUCTION.

The necessity for a treatise on Indian Water Supplies has been apparent for a long time. Books on waterworks, which deal mainly with European and American practice, fail to supply information that is essential to Engineers and others engaged in designing, constructing, or managing waterworks in India and the East. Rai Sahib K. C. Banerjee, who has had a long experience of this class of engineering that fully qualifies him to deal with the subject, has set himself in this volume to supply both the practical and the scientific information that is lacking in other books.

Mr. Banerjee has succeeded in compiling the most comprehensive treatise on Indian Waterworks that has as yet appeared. His comparisons with waterworks in other countries are very instructive, whilst the information regarding installations in India, and more especially in Bengal, is very complete, and includes facts and statistics which are now published for the first time. This information must be of value to Indian Engineers, members of local authorities and waterworks managers.

The long professional connection between Mr. Banerjee and myself, which commenced more than 20 years ago, adds to my interest in his valuable and interesting work and makes it a pleasure to me to recommend it to all who are concerned with the subject.

G. BRANSBY WILLIAMS.

28, VICTORIA STREET,
WESTMINSTER, S.W. 1.
May 17th, 1930.

PREFACE.

This book is a development of the course of lectures delivered at the Bengal Engineering College. The lectures were presented to the classes in a form found suitable to the author's plan of studying the subject. Among the many books dealing with Water-Works Engineering the author has found none designed to cover the entire subject as applicable to the conditions prevailing in India. The need for a single book of this character has been expressed by his associates in the profession and by the students in the class room. This book endeavours to fulfil the requirements of both.

The ground covered, includes the interpretation of the principles and the methods of prelimenary investigation, designing, construction and maintenance of Water-Works as practised in this country. More attention has been paid to the fundamental principles of the subject rather than to the details but at the same time illustrations have been freely given from works carried out.

This work has been to a considerable extent a compilation of the notes and quotations which the author collected during his years of practice and teaching the subject. References have been made, as far as possible to the numerous sources from which the information has been derived. In a few instances however the original reference have been missed or the phraseology altered in the lectures in such a way as to make it impossible to trace the original source, so that it may be, in some cases proper acknowledgement has not been credited, for which the author begs to be excused.

The author is indebted to many of his friends in the profession for valuable suggestion in connection with the preparation of this book, and particularly grateful to Mr. G. B. Williams, M.I.C.E. late Chief Engineer, Public Health Department, Bengal for kindly going through the

manuscript. I am also indebted to Mr. S. A. Bunting, M.I.C.E. and Dr. S. N. Sen, meteorologist for many valuable informations furnished by them in connection with the pumping machinery and meteorology of India respectively.

K. C. BANERJEE.

CALCUTTA.
The 25th. March, 1932.

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INDIAN WATER WORKS PRACTICE

CHAPTER I.

DEVELOPMENT OF PUBLIC WATER SUPPLY.

Early development:—Of food, air and water, the three essentials for the subsistence of life in plant and animal, water is by far the most important. It is necessary for the proper performance of the physiological functions in all living organisms in this world and constitutes nearly two-thirds of the total weight of human body. It enters into their systems either as a circulating fluid or as a vehicle for nourishment or for the supply of oxygen necessary for converting food into vital energy or for removing the waste products.

The necessity for finding sources of pure drinking water was naturally felt by man in his primitive stage. Early settlements in all countries have therefore been made near springs, lakes and rivers where plenty of wholesome and transparent water could be obtained. These settlements being few and far between, and the number of people living in them being limited, natural sources of supply were found to be sufficient to meet their requirements which were mainly for domestic purposes.

The earliest method of artificial water supply amongst the ancients was doubtless from wells, mention of which can be found in our *Rig Veda* which dates from 4000 year B.C. In the *Ramayana, Mahabharata* and other *Puranas* of the Hindus mention of wells can be found in several places. In the *Arthashastra* it was stipulated that there must be provision of a well for water supply for every ten houses. Besides India, wells as sources of water supply were not uncommon in other countries, such as Greece, Egypt, Assyria, etc. Joseph's well at Cairo is one of the most famous wells excavated in rock to a depth of 297 feet below ground. The Chinese understood

the process of sinking wells ; some of their wells are 1,500 feet deep and the process of sinking was almost identical to that still practised in China.

With the increase of population the people began to congregate in towns and cities, so the limited supplies from wells or springs were found to be quite inadequate. In consequence, the works necessary for the conveyance of water from a distance and storage in the neighbourhood of towns became a necessity. As regards the works of storage, even at the time of the occupation of India by the British there were no less than 5000 impounding reservoirs in Madras Presidency alone. Some of the earthen embankments thrown across the rivers are nearly 30 miles in length. Ruins of aqueducts can be found in different parts of this and other countries ; their construction excites the admiration of the waterworks engineers of the present day.

The most important of such works in the West are those constructed by the Phœnicians. In Cyprus, water was supplied to temples by rock-cut underground conduits carried across intervening valleys in syphons. Such conduits have been found in Citium, Amathus, etc. In Jerusalem, a syphon system was also adopted in the lower ground near Rachel's tomb where the pipes made of large pierced stone embedded in rubble masonry are still in existence. The earliest form of aqueduct in Greece was discovered in the Island Cos beside the fountain Burinna on Mount Oromedon. It consists of a bell shaped underground chamber on the hillside to collect water from a spring ; from the reservoir the water was taken through a channel 114 feet long and 6·5 feet high. Later in the 7th century B.C. Eupalinus, a Greek Engineer for the water supply of Magara, cut a tunnel 8 ft. by 8 ft. through a hill at Samos. This tunnel was 4000 ft. long and contained within it a channel 3 feet wide leading to baths, fountains etc. in the town.

E. H. D'Avigdor in his article on "Waterworks in ancient Rome" says :—"The Romans possessed three almost independent services for which they used water of different degrees of

purity. The least clear and most loaded with sand, such as the Anio aqueduct supplied, was used for public baths, watering streets ; the clearer water from Tepula and Alsietina served for tanks and fountains, washing troughs ; while the very best (Virgo, Marcia and Claudia) was confined to drinking purposes". These springs remained undefiled even after the heaviest rains. The aqueducts of three different groups mentioned above were 19 in number and built in the fourth century B.C., the total length of these aqueducts was nearly 400 miles. The water from these used to flow into small tanks built in different parts of the city for intercepting silt and other suspended matter in the water. From them water was distributed into cisterns, public fountains and private residences.

Distribution through pipes was probably unknown to the ancients. London was perhaps the first modern city in the world which at the end of the 16th century used lead pipes for conveyance or distribution of water. After this, for many years wood pipes bored out of logs came to be used and in some parts of Europe such pipes are still in use. In about the year 1800 cast iron pipes were brought into general use for water supplies in towns and cities. Recently it has been discovered that cast iron pipe, laid 250 years ago in Varsailles, is still in working condition.

The next stage of development was the use of pumping machinery for lifting and distribution of water. In this respect also London possessed (in 1582) the first pumping plant for the supply of water from the Thames through lead pipes to the city. The application of steam to water pumping in the 18th century gave a considerable impetus to the progress of water-works engineering. A steam pump was installed in London in 1761, in Paris in 1781 and a second in London in 1787. The growth and development of waterworks plants really date from the 18th century but not until the latter half of the 19th century was very rapid progress made.

In India water supply on modern lines is of comparatively recent development. The first work was for the supply of water to Calcutta which was completed in May 1870 and before

1900 no less than fifty more waterworks were brought into operation.

Development of the Art of Purification :—The earliest and the most widely adopted method of securing good water was doubtless the use of ground water (*i.e.* water from wells or springs) which by its passage through porous ground has been almost completely relieved of its suspended impurities. Gradually with the increase of population and civilization, when these sources near settlements were found inadequate, resorting to surface water became a necessity. Surface waters conveying suspended impurities, were mostly turbid and consequently unattractive as drinking water. It must have been noted by the ancients that a turbid water, if allowed to remain quiescent for some time, lost almost all its suspended impurities by settlement and thereby its appearance was improved to such an extent that it might be used for drinking purposes. This led to the construction of impounding reservoirs which have been used both for water supply for drinking purposes and irrigation in this and other countries.

Clarification of water by settlement was the earliest step in the art of purification and is still a very important and efficient one. It is now however usually regarded as a preliminary to more complete purification. The straining action of sand and gravel was probably observed by the ancients in India and other countries in the same way that clarification effected by settling tanks had been. Works depending upon this natural action are not uncommon in many of our ancient temples. The practice of obtaining water by digging a hole in bed of sand alongside a flowing stream in many districts in our country is also an instance of this method of purification. The application of the principle to public waterworks is however quite recent. Sand filters were constructed in the 19th century first in England and then on the continent of Europe. The first sand filter was constructed by James Simpson for the London waterworks in 1829 and gradually the system was extended afterwards to other cities on the continent.

The art of coagulation was also known to the ancients in

this country. Before the *Tantric age,* they used solutions of certain vegetables such as *Nirmali* for such purpose and latterly alum was used as a coagulant.

The next stage of progress was the introduction of the mechanical filter for public waterworks. The considerable cost of sand filters and the space necessary for their accommodation led to the invention of mechanical filters with an artificial filtration layer made by the action of alum on the alkaline earths present in the water.

The last stage of development is the sterilisation of water. The chlorination of water is now recognised both in Europe and America under certain conditions to be a practical, efficient and economical agent for water purification. Besides chlorine and its compounds, water has been sterilized by the action of ozone, ultraviolet rays and by what is known as the "excess lime" process. This phase of water purification is now well established and marks an important stage of progress in the practical development of the art.

Indian Sanitary Laws in regard to Water :—The Rishis of India considered water to be a sacred gift of God and ordained that it must be kept free from any pollution; if by accident any source of water supply would be polluted, they laid down the process of purification. Unlike the people of our age who think that they are at liberty to pollute any source of water to any extent they liked, with the belief that it is only necessary to purify the water that may be required for human consumption, the ancients held all sources of water as sacred.

In the *Aranya Khanda of Yajurveda* we find the following commandments with regard to water :—"Do not spit out with retching in the water. Do not pass urine or discharge excreta into water. Do not drop blood into water. Do not throw hair or nails, bones or ashes, nor dip dirty clothes into water. For to do so is to abuse a precious gift of God and discharge them."

In *Jajnavalka Sanhita* the use of the following eight kinds of water is prohibited, *viz.* : (1) water kept boiled by a stranger, (2) foaming water ; (3) heavy turbid water ; (4) water giving

offensive smells ; (5) water rising in bubbles ; (6) hot water ; (7) water containing germs and (8) salt water.

In *Ramayana* we find Prince Bharat saying that he who gave consent to the exile of Rama was guilty of defiling water and was fit for the infernal regions.

In *Mahabharata* again we find Arjuna swearing that if he returned to camp without killing Jayadhrata on the field of battle, his place would be in the same infernal region where men, who did spit out retching or discharge urine or excreta in water, were doomed.

Sathathapa prohibits the bathing in a tank defiled by bathing or washing by persons suffering from the following diseases, *viz.*: (1) eye-sore ; (2) itch on head or ear or other parts of body ; (3) epileptic fits ; (4) running of nose or ear ; (5) consumption ; (6) leprosy, small pox, diarrhœa and other contageous diseases.

Jajnavalka also prohibits the use of the following waters. Water that remains after the washing of one's feet and private parts of human body or after the drinking by another person. He also enjoins that water from tanks where the washermen, butchers, chucklers and women after their menses or child-birth use or bathe must not be used.

In *Manu Sanhita* we find as follows :—"Let him not cast into water either urine, saliva nor cloth or any other thing soiled with impurity, blood or any other kind of poison."

In *Bhabaprokash* water is classified into two groups, *viz. Dibya* and *Bhowma* ; the former is water obtained direct from the sky and the latter means water that is obtained after precipitation. *Dibya* water is again sub-divided into four different varieties *viz. Dharaj, Karaka-bhaba, Tausara* and *Haima.*

Dharaja water is that which is collected directly as the rain falls, through clean cloth and held in clean vessels of stone, gold, silver, copper, glass or earthen pots. This water is considered to be the most health-giving water. *Karakabhaba* is the water derived from hail-stones as they fall on the ground and collected in the same way as above. It was not considered so suitable as the former. *Tausara* water is that derived from

molten snow and *Haima* is that obtained from dew, fog or mist. The waters from these two sources are not considered satisfactory, as they produced many diseases when used by men. *Bhowma* water is classified into three groups, *viz. Jangala, Anupa* and *Sadharana.* The first two classes of water are those obtained from forest and marshes and they were considered unhealthy. The last variety is sub-divided into various classes ; of them the following are the principals :—

1. *Nadeya*—the water obtained from a flowing river. This water is considered to be suitable for domestic use in certain seasons of the year.
2. *Udvida*—springs flowing in streams after piercing the ground. This water was considered healthy.
3. *Nairjhara*—water obtained from a waterfall was also considered to be beneficial to health.
4. *Saraga*—water from an impounding reservoir on the side of a hill was considered suitable for drinking purposes, if the area of precipitation was free from fæces and urine of men and animal.
5. *Taraga*—is a tank more than 2000 cubits in length ; the water of this was considered satisfactory within certain limits.
6. *Koupya*—well water was considered to be very sanitary.
7. *Choudya*—water from a natural cavity in bed of sand or pebbles was also considered to be healthy.
8. *Bidira*—the water coming out of sandy stratum near a river was considered to be very refreshing and healthy.

In this book are also given details of instruction as to the seasons of the years in which the water from these sources can be used for domestic purposes. Regarding purifying water accidentally polluted, the following process is given in *Ayurveda.*

"The impure water is to be boiled or heated in the sun by putting into highly heated gold, silver or earthen vessels and then cooled ; this process was to be repeated seven times.

Water purified in this way was enjoined to be used with camphor after passing it through a thick sheet of cloth."

In the *Auosutra Sanhita* also the following instructions are given :—"It is good to keep water in copper vessels, to expose it to sun light, and to filter it through charcoal."

Water and Propagation of Diseases :—The bacterial origin of infectious diseases is no longer regarded as a theory but as a certain fact, and the whole weight of public opinion now gravitates towards the one central aim, *viz.* bar the avenues by which these enemies of health approach the individual, and the health of the community will be safeguarded. The discharge of liquid wastes and sewage at the present day into rivers, lakes and other sources of supply gives ample opportunity to disease germs to enter our system through public water supplies. The diseases that are generally propagated by these germs are those peculiar to the intestinal tract of human body ; hence cholera, typhoid, dysentery and gastro-intestinal disturbances are generally regarded to be derived from polluted drinking water wherever these diseases occur as epidemic or endemic. It may be noted that these diseases may be spread in many other ways but water is generally traced to be a very favourable vehicle for propagation. If the theory of Sedgwick and Mac Nutt, that inflamatory diseases of the respiratory organs may be also to a certain extent water-borne, is accepted, the polluted sources of water supply may be held responsible for another group of diseases prevalent amongst all classes of people.

Probably no disease is more freely characterised as "water-borne" than cholera. An epidemic outbreak of this disease is not uncommon and is often traced to polluted sources of water.

In 1817 a violent outbreak of cholera in Jessore rapidly spread over different parts of India. Its ravages continued unabated for three years and then it began to spread into China and Persia and in 1823 it reached Asia Minor and Russia. For the next 7 years it did not advance westward any further but a fresh outbreak in 1830 in Russia caused the disease spread over the latter country and into northern Europe and the British Isles. During the next 5 years it spread southward

invading north of Africa. The second outbreak in India and China in 1841 spread to Europe in 1847, and the third began in 1850 and entered Europe in 1853, thence it crossed the Atlantic and invaded North and South America where it was particularly severe. In 1884 Toulon was invaded by another epidemic of cholera said to have been brought by a troop ship from Cochin China. This was said to be the greatest cholera epidemic in Europe, according to Dr. A. J. Wall, no less than 250,000 people having died in Europe and at least 50,000 in America. France and Italy suffered very heavily, the former alone having lost 120,000 souls. In this way the disease has spread all over the world and caused deaths by thousands. In 1883 the German Government sent Koch, a German scientist to India specially for the purpose of investigating as to the cause of the disease. He discovered the *comma* bacillus in the stools of cholera patients. This bacteria is generally now considered to be the cause of infection.

On the other hand, Europe is the home of typhoid fever from which it spread to the other parts of the world. Like cholera it has victimised thousands of persons in different parts of both the Eastern and Western continents. In 1880 the organism which caused this disease was discovered by Eberth in the spleen of persons dying from typhoid. It is now generally accepted that epidemics of typhoid fever are due to these germs of which water is one of the vehicles of transmission.

As polluted water on the one hand is held responsible for the transmission of pathogenic germs and for causing epidemics of water-borne diseases, pure water on the other hand decreases diseases other than cholera, typhoid etc. This phenomenon has been recently studied by Sedgwick, MacNutt and Hazen in America and Dr. Reincke in Germany. Prof. Sedgwick in a paper collected numerous statistics of mortality and has termed the coincidence between lowered death rate and pure water supply as the "Mills Reincke Phenomenon". Allen Hazen in another paper gave a quantitative expression to this phenomenon thus "where one death from typhoid fever has been avoided by the use of better water a certain number of deaths

probably two or three from other causes has been avoided". This ratio will vary very widely in different towns and countries owing to the local conditions being different. In Hamburg the ratio was found to be 1 : 15·8 ; in Lawrence (Mass) 1 : 4·4 ; in Lowell (Mass) 1 : 6 ; while in Binghampton N. Y. it is only 1 : 1·5.

In India this phenomenon has not been studied yet. Owing to the absence of separate statistics of mortality from typhoid fever it has not been possible to give calculations with regard to towns where pure water supply is being given. All that can be said however which will be evident from the diagrams Figs. 1, 2, 3 and 4 is that there has been continual decrease of

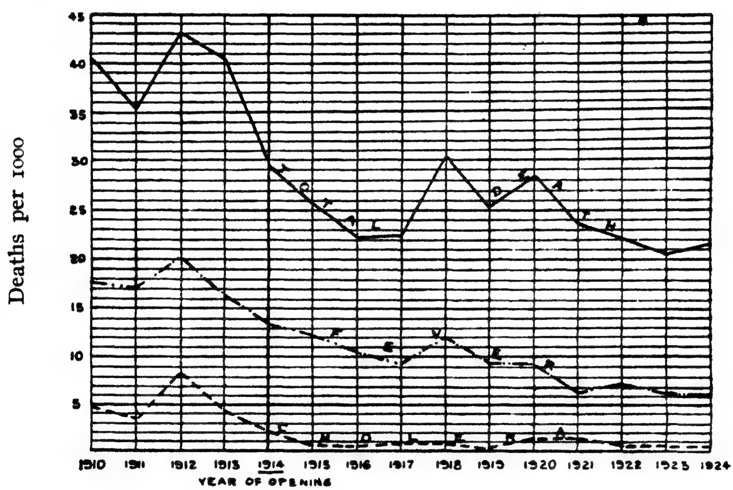

Fig. 1. Diagram showing reduction of mortality by introducing pure water supply at Hooghly-Chinsurah.

mortality from fever (which is mainly malarial in this province) besides that from cholera and other water-borne diseases. This is another instance in support of the "Mills Reincke phenomenon". The reduction of death rate from fever after the introduction of pure water supply has been so much that one may be led to believe that it may be possible to cripple malaria by the construction of works of pure water supply throughout

the province. The temporary rise in mortality during 1918 and 1919 in the diagrams was due to influenza epidemic.

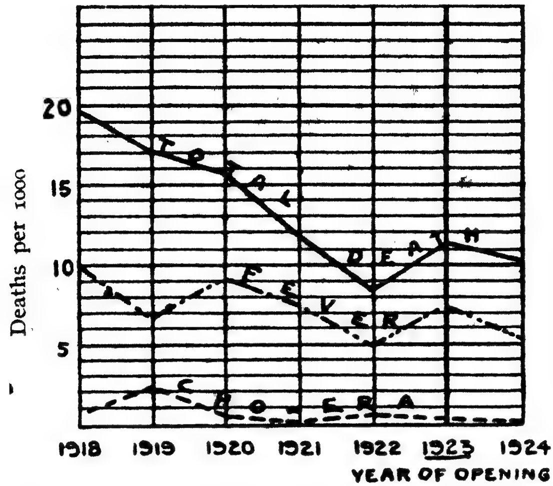

Fig. 2. Diagram showing reduction of mortality by introducing pure water supply at Krishnagore.

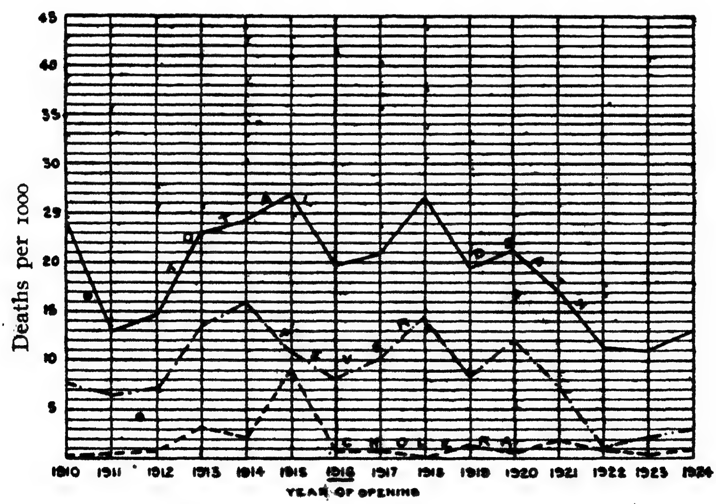

Fig. 3. Diagram showing reduction of mortality by introducing pure water supply at Bankura.

The reduction in the incidence of water-borne disease

caused by the introduction of a pure water supply to a community depends not merely on the bacterial or chemical

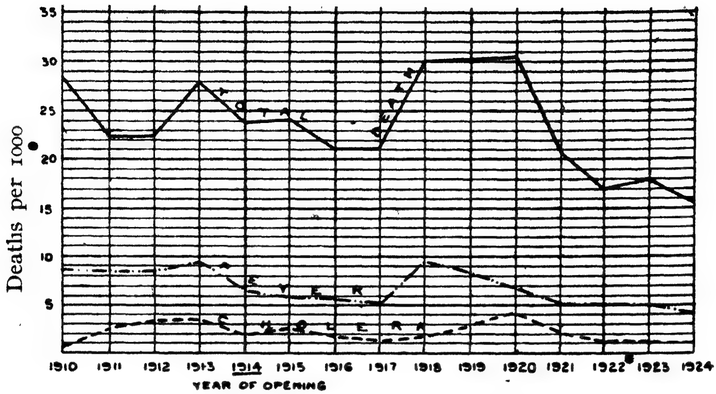

Fig. 4. Diagram showing reduction of mortality by introducing pure water supply at Serampur.

purity of the supply but considerably on the method of distribution. To produce an ideal effect, the water should be distributed in such a manner that a consumer may draw it direct from the taps on his service pipe, clear, cool, sparkling and in the same conditions as it leaves the street mains, and the supply should be abundant and never-failing. This obviates the necessity of storage on the premises of the consumer which can under the most favourable conditions deteriorate and is liable to pathogenic infection.

In 1921 a comparison of the effects of the introduction of filtered water supply in different municipalities in Bengal was made by Dr. Bentley which showed that at Hooghly Chinsura, the percentage of reduction of cholera was 88·8 in 6 or 7 years after the introduction of pure water supply. This was found to be the maximum reduction in the province. This result was largely due to the fact that the Municipality (in addition to the advantage of a pure water supply) had the longest number of distribution pipes and the greatest number of standposts per 1000 persons of the population served. Consequently pure water was comparatively more easily within the

reach of the consumer than it was anywhere else in the province. At Serampur the percentage of reduction was 64·9, the comparative lower figure was due to the Chatra Ward being out of the zone of supply at the time and therefore the incidence of death-rate from cholera for the whole town was not a correct indication of the effect of water supply. In the case of Bankura, the reduction was 70·3 per cent with one-third the length of pipes per 1000 persons. This was due to the peculiar position of the populous portion of the town in reference to its old source of supply. Nearly the whole of the population used to draw the water required for domestic purposes from the river Gondheswary. The new water works having provided a line of pipe on the ridge of the town, and within half the distance they had before, the people generally began to draw water from the standposts ; only those who live close to the river perhaps get their supply from the river and thereby the vital statistics are also affected to a certain extent.

CHAPTER II.

QUANTITY OF WATER REQUIRED BY A COMMUNITY.

Variation in Character of Demand :—The water supplied to a community must not only be good in quality but also be sufficient in quantity to meet all requirements without inconvenience or discomfort. The requirements vary according to the geographical position, social habits and customs, and state of civilisation. It increases with the increase of population and consequent development of industry, wealth and refinement. In the same city water actually consumed varies very widely in different houses, the poor drawing water from standposts gets only 3 to 5 gallons a day, a middle class citizen having metered connection will easily consume 15 gallons a day while in an unrestricted and unmetered supply the consumption may go up to anything between 50 to 100 gallons per head per day. The consumption will also vary from month to month, week to week and day to day ; it is generally maximum in the hottest part of the year and also in the early hours of the morning.

The requirement of supply to a community in this country is generally expressed in the terms of average number of imperial gallons of water consumed per head of population per day throughout the year.

The total average daily supply of a town is found out by multiplying the average daily supply per head per day by the population served.

The first factor is estimated in two ways, *viz.* by obtaining an approximate figure of consumption from a similar city, or by ascertaining the different purposes for which the supply is necessary and what quantity is likely to be used under each item of supply.

To estimate average daily supply by the first method, it is necessary to obtain statistics of water supplies in different towns

in the district of the same size, and reduce them to some general average that will meet the requirement of the case under consideration. The statement below gives the actual quantity supplied to the principal towns of Bengal, Behar and United Provinces during last decade :—

TABLE I

	Population.	Supply in Gall. per head per day.
Calcutta	1,077,264	36·00
Barisal	26,741	3·70
Berhampore	26,670	9·23
Burdwan	32,000	12·29
Dacca	120,000	13·22
Howrah	195,300	17·40
Mymensingh	25,320	14·06
Darjeeling	20,935	24·84
Khulna	8,000	3·20
Chandpur	6,000	1·99
Jessore	8,000	4·75
Hooghly-Chinsura	29,937	19·10
Bankura	8,470	7·57
Chittagong	36,074	11·04
Krishnagar	19,013	6·42
Arrah	40,769	4·60
Bhagalpore	68,878	14·50
Gaya	67,562	15·50
Monghyr	46,825	6·10
Muzaffarpur	32,755	5·10
Patna-Bankipur	25,200	14·90
Agra	185,946	24·54
Allahabad	157,220	22·53
Benares	198,447	21·91
Cawnpore	216,436	25·20
Lucknow	201,334	18·28
Muttra	38,400	25·33
Bombay	1,274,150	49·50

Mr. G. Bransby Williams in a note on the subject written 15 years ago estimated the then average daily consumption of water as follows:—

TABLE 2

Towns with	3,000	to	5,000	population	2	to	3	gallons.
,, ,,	5,000	,,	10,000	,,	3	,,	5	,,
,, ,,	10,000	,,	20,000	,,	8	,,	10	,,
,, ,,	20,000	,,	50,000	,,	12	,,	15	,,
,, ,,	50,000	,,	200,000	,,	18	,,	25	,,

Since Mr. Williams investigated this subject there has been a tendency for the consumption of water in Indian towns to increase. The rate above would be considered too low for many towns to-day.

Figures primarily intended for waterworks practice in Bengal will be found equally applicable for Behar & Orissa and other Provinces. It may be noted here that the gradual increased rate of consumption in these five classes of towns is due to the corresponding increase in the number of house-connections and to the increase of water utilised for other purposes such as street watering, drain and sewer flushing etc.

The other method of estimating the average daily consumption per head of population is by finding out the different purposes for which the water is likely to be utilised and then ascertaining from actual practice the quantity of water consumed for each purpose. The following are the various purposes for which a public water supply is utilised and the quantity actually required for each purpose in this country.

TABLE 3

Household.

Drink	0·50 gals. per head per day.
Cooking	1·00 ,, ,, ,, ,, ,,
Ablution	4·00 ,, ,, ,, ,, ,,
Utensils and house-washing	3·0 ,, ,, ,, ,, ,,
Washing clothes ..	3·0 ,, ,, ,, ,, ,,
Water-Closets ..	6·0 ,, ,, ,, ,, ,,

Municipal.		
Road-watering	..	1·5 gals. per head or 1·0 gal. per 100 sq. ft. of road surface.
Sewer flushing	..	0·5 gal. per head per day.
Fire prevention	..	0·2 ,, ,, ,, ,, ,,
Cattle and Animals.		
Horse ..	..	15 gallons each.
Pony or Mule	..	6 ,, ,,
Domestic Cattle—		
Cow ..	..	8 ,, ,,
Sheep or Pig	..	1 ,, ,,
Two-wheeled carriage		8 ,, ,,
Four-wheeled carriage		15 ,, ,,
Watering Gardens	..	¼ gallons per sq. yd.
Ordinary bath ..	..	30 to 40 gallons.
Bath tub ..	..	6 to 8 gallons.
Shower bath ..	..	3 to 6 gallons.

Besides these an allowance must be made for loss and wastage, the causes of which are bad plumbing, leaky water mains, and waste due to careless and lazy habits of the consumers.

When with the object of preventing waste in houses all connections are metered, and liberal allowance is made for water used for public, there is still a large amount of water apparently supplied which remains unaccounted for. This unaccounted waste varies very widely in different waterworks. In English waterworks, the waste is estimated at 15 to 20 per cent on the average supply, while in America it ranges from 20 to 45 per cent. There is no reliable figure of waste in this province.

With regard to the waste from leaky mains, American engineers generally assume a maximum allowable leakage of 50 to 70 gallons per mile per inch diameter of pipe.

Estimation of Future Population—Population is a term employed for the total number of human beings living within a certain area at a given time. This is ascertained in every

civilised country by means of a census recording periodically the number of people found in it on a certain date together with the details of age, sex, nationality, religion and conjugal condition. The first of this kind was recorded in India in the year 1872, and then in 1881, and since that year it is being enumerated every ten years. These census statistics are published by Government of India for different provinces and give details of population of every district, every municipal town and cities in India. These operations are not perfect either in scope or in accuracy, but they are the only data on which the prediction of future population can be based.

Fundamentally, the population in certain area should increase in regular geometrical progression when the number of births exceeds that of deaths and the ratio of birth and death to the population remains constant, and also that there is no movement of population. But there is hardly any locality in which all these conditions exist, hence several methods of approximation have been devised to predict future population. Of these the following two graphical methods are generally considered sufficiently reliable and can be used with certain circumspection.

The simplest method most commonly adopted is to plot the numbers of population in past census records on a piece of section paper ; the numbers of population, and the years of record are taken as ordinate and abscissa respectively. The line of curve so obtained is produced in continuation of the apparent trend of the plotted curve. In this method it is assumed that the future growth will be similar to that in the past, but this can rarely be correct in any place. This method is useful for short period estimates.

The other method is the one in which semi-logarithmic section paper is used in place of ordinary section paper. In this case, the census figures are plotted on the side which is divided into logarithmic scale and the years of census on the side divided in ordinary scale. The curve is produced in similar way as before. A series of numbers increasing at a constant rate when plotted in this kind of paper will be a

straight line. When the curve of the census figures of a town plotted in this paper is straight, a prediction of the future population can be easily made by continuing the straight line. But this is seldom the case. It will usually be found that the census figures do not give a straight line, but a curve sloping upwards or downwards. In the former case, the estimate can be based on the production of the curve but this is hardly possible in the latter case; with the introduction of a water supply scheme, the decrease of population will be arrested, and within a short time there is every likelihood of the population being increased. Consequently, engineers often in such cases secure data from older and similar sized community where the conditions are reasonably comparable and base their estimates on the figures of growth of that city.

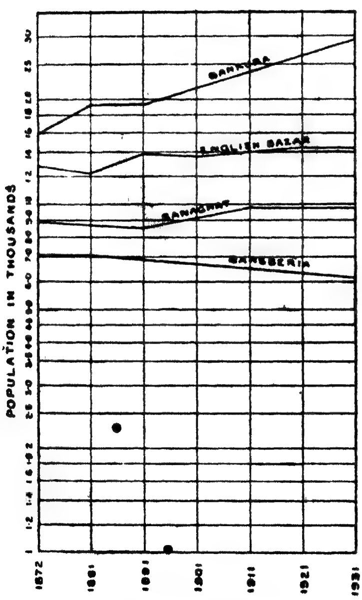

Fig. 5.

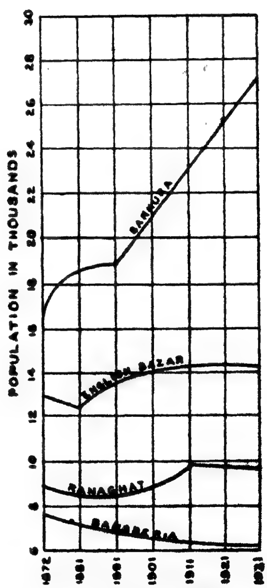

Fig. 6.

Diagrams shewing Graphic methods of estimating population.

The total population obtained by either of these graphical methods gives only mathematically the approximate figure

which requires further modification for other circumstances, such as density of population, state of industries and their development, character of housing, transportation &c.

The fundamental principle of the growth of population is economic as well as biologic, and a knowledge of these laws and of the movement of population is a valuable guide in making estimates.

Four examples are given in the accompanying diagrams (Figs. 5 & 6) showing how the problems are solved by the two graphical methods described above, and the figures obtained by them.

Variation in Supply :—The data previously given relating to average daily supply have reference to annual quantities reduced to daily average. The daily consumption is not uniform throughout the year but at times it is in excess of the average and at other it falls below it. The maximum monthly consumption is from 10 to 25 per cent in excess of

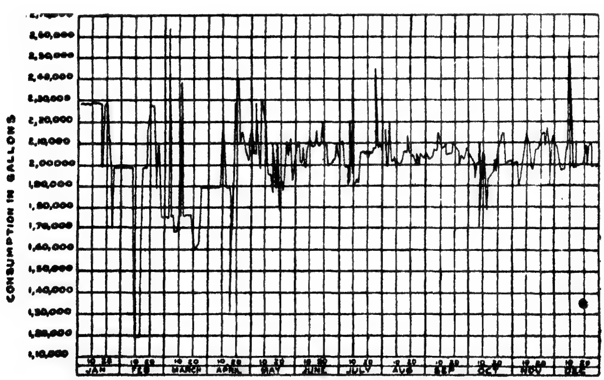

Fig. 7—Diagram shewing daily variation of supply, Mymensingh.

the average, and the maximum daily supply is from 30 to 40 per cent more than the average supply per day, while the maximum hourly supply is from 50 to 100 per cent in excess of the average hourly supply of a particular day. The ratio of maximum hourly supply to the average hourly supply varies very widely in different towns in India. It ranges from 2·25

to 2·50. This is an important factor in the design of water works and its distribution system.

The diagrams Figs. 7, 8, 9 give the daily, hourly and monthly variations in Mymensingh waterworks.

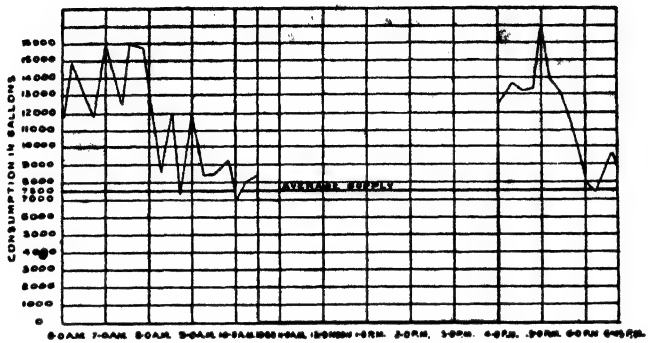

Fig. 8—Diagram shewing hourly variation of supply, Mymensingh.

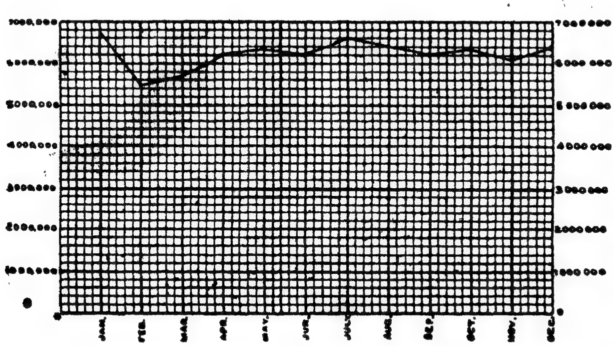

Fig. 9—Diagram shewing monthly variation of supply, Mymensingh.

CHAPTER III.

SOURCES OF SUPPLY.

Classification of Sources of Supply :—The ultimate source of all water supplies is the aqueous vapor in the atmosphere including dew, mist, snow, rain or hail, which reaches ground and it is the physical and topographical character of the surface on which it falls that determines the future course of the moisture and consequently of the source of supply.

When rain or hail falls on the surface of the earth, a part of it is absorbed by the soil or vegetation and is lost, another part sinks deep into the ground to form underground reservoirs for the sources of galleries and wells, or to appear at the surface again at a lower level in the form of springs ; a third portion is returned direct to the atmosphere by evaporation either from land or water surfaces. The remaining portion runs off the surface to the drainage lines forming streams, rivers or lakes.

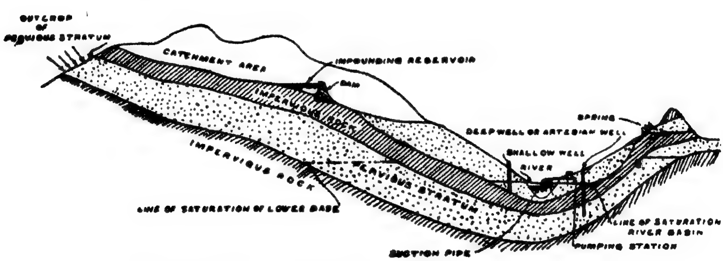

Fig. 10—Different Sources of Water Supply.

Broadly speaking, the sources of supply may be divided into two classes according to whether the supply is derived from the water falling on the surface or after it has penetrated into the ground. The former source of supply is known as surface water supply and the latter ground water supply. The surface water may be intercepted and stored (1) in tanks or

cisterns suitably constructed to receive the rain water from roofs of houses; (2) or may be obtained direct from streams or rivers, or natural lakes; (3) or from impounding reservoirs.

The ground water supplies are obtained from (1) galleries, (2) shallow or deep driven wells and (3) from artesian wells and springs. Fig. 10 shows the different sources and how they are utilised.

The selection of a source from which to obtain supply for human consumption depends upon a variety of considerations among which the following are important:—

1. Purity of supply.
2. Volume and permanency.
3. Elevation with regard to the place to be supplied.
4. Nature of the intervening ground.
5. Financial capacity of the local body.

Purity of Supply:—Water does not exist in nature chemically pure; owing to its almost universal solvent power, it holds in solution many foreign matters through which it flows. For convenience, the impurities in water may be grouped into two classes according to the sources from which they are derived. Water during its course of flow over the surface or through the ground dissolves many mineral matters or holds them in suspension. Provided no contact of the water with animal life or activity has occurred, the acquired impurities may be termed *inorganic impurities* in distinction of those derived from sewage, manufacturing wastes, decomposition of animal or plant. Waters polluted from the latter sources are said to contain *organic impurities* and are unfit for human consumption without purification.

(*a*) THE INORGANIC IMPURITIES:—These are divided into two classes, *viz.* (1) minerals held in suspension and (2) those held in solution.

Iron, magnesia, chlorine in the form of chloride of sodium or common salt, sulphuric acid as sulphate of magnesia, sodium, potassium or lime, carbonic acid either in free state or in the form of carbonate of lime or magnesia are the most commonly found mineral matters in solution in natural waters.

Clay, fine sand and other constituents of soil varying in size from 0·00001 inch dia. to coarse sand or gravel are usually found in suspension in water flowing over the surface. Rarely, deleterious metals like lead, arsenic, copper and zinc are found in solution.

Iron is not harmful when occuring in drinking water in a comparatively large amount ; but as 0·05 part per 100,000 of water gives a bad taste, no more than this should be allowed to remain in any public water supply. Iron is derived by water when flowing over the iron deposits on the surface or iron ores of deeper strata or from the decomposed vegetable matter. Most vegetable matter contains more or less some iron, the chlorophyl, the green substance on the leaves and plants contain about ·01 to ·06 per cent of iron.

Lime and magnesia occur generally in water as carbonates, sulphates or other soluble salts ; of these carbonates cause *"temporary hardness"* and the other salts *"permanent hardness."* Carbonates are caused by the carbonic acid contained in the water dissolving lime or magnesia when passing over or through deposits of the same. When water is boiled, carbonic acid is expelled leaving lime and magnesia in insoluble state. The quantity of the above matter deposited on boiling constitutes what is termed *temporary hardness.* On the other hand, the sulphates, chlorides and nitrates of calcium and magnesium are soluble in water and therefore are not precipitated on driving off carbonic acid. They remain in solution and form *permanent hardness* or hardness after boiling. For the purpose of comparison, both the kinds of hardness are recorded in the terms of amount of the dissolved carbonate of lime that would decompose and precipitate the same amount of soap. English degrees of hardness—Clark scales are recorded in grains of calcium carbonate per imperial gallon to the soap destroying power of water. Each degree of hardness is equivalent to 0·56 of calcium oxide (CaO), 0·74 of slaked lime $Ca(OH)_2$ 1·06 of anhydrous sodium carbonate (Na_2CO_3) or 2·86 of crystallised sodium carbonate ($Na_2CO_3, 10H_2O$).

The following conversion table of hardness to different standards may be of use:—

TABLE 4

Parts per 100,000	..	..	1·00 per 100,000
Clark Scale ..	..	..	1·45 ,, ,,
French degrees	..	..	1·00 ,, ,,
German degrees	..	..	1·78 ,, ,,
Grains per gall.	..	..	1·71 ,, ,,

Statistics as hitherto collected by different authorities show that a supply of hard water does not produce any appreciable effect on mortality, although slight intestinal trouble may be caused by the change from soft to hard or from hard to soft water. On the other hand, very soft waters being less potable are not wholesome. On the whole a moderately soft water is most hygienic.

Investigations in France have led scientists to the conclusion that certain quantity of mineral salts is necessary for bodily nutrition and the use of demineralised water often produce tubercular conditions in the consumers.

The degree of hardness of water in different countries varies very widely. In England it varies from 1·70° to 23° on Clark scale. In Bengal the total hardness generally ranges from 3 to 20 parts per 100,000 of water and in rare cases more than 30.

Hard water is not suitable for boilers, textile industries, breweries, distilleries, bakeries or for the purpose of cooking.

Sulphates of sodium and magnesium occur in small quantities in some waters and cause foaming in boilers but no deposits. Sulphate of magnesium on the other hand forms boiler scale and sulphuric acid is set free as a corrosive agent.

Chlorine is usually present in water in the form of a common salt, the chief source of which is the ocean from which it is carried by evaporation and held in solution by the rain. The amount of chlorine in rain water varies in different districts. This is known as *"normal chlorine"* of water in the locality. Besides this source, chlorine is also derived from the urine of

men and animals. The amount of chlorine present in any rain or surface water is neither unpalatable nor injurious to health, nor harmful as feed water to boiler. It may not however be used for certain manufacturing purposes, but when water in certain locality contains more chlorine than normal, it should be looked upon with suspicion until further evidence is adduced that the source of increase is other than excreta of men or animals.

The following table gives the taste of drinking water according to the parts of common salt present per 100,000.

TABLE 5.

Parts of common salt present per 100,000.	Taste.
Up to 40	Imperceptible.
From 50 and above ..	Brackish.
100 to 250	Strongly saltish but bearable.
250 to 500	Unbearably saltish.

Lead, zinc and arsenic when present in water in comparatively large quantities may cause serious poisoning. There are instances when lead and zinc dissolved from service pipes by the action of water containing high carbonic acid have caused accidents.

(*b*) ORGANIC IMPURITIES:—The really dangerous matter present in the water from the sanitary standpoint is its organic impurities, the correct estimation of which being of prime importance. These may be of either vegetable or animal origin and vary in size from the smallest bacteria to a large fish. The organic impurities may be present in water in either of the following three states, *viz.*:—

(*i*) Living organisms;

(*ii*) Products of living organisms, such as albumen, urea, tissue etc. in suspension or solution;

(*iii*) Products of decomposition of organic life such as salts of ammonia, carbonic acid and nitric acid.

Organic impurities cannot be directly determined by chemical analyses but can only be judged in an indirect way.

It is well known that every organic matter contains amongst others in one form or another the three elements—carbon, nitrogen and hydrogen of which carbon and nitrogen oscillate between the organic and the inorganic states. As soon as an organism loses its life, decomposition sets in first by oxidation of carbon, leaving nitrogen and hydrogen combined in the form

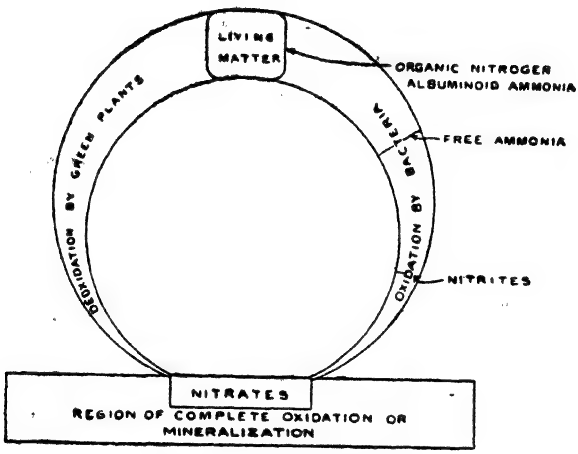

Fig. 11—Nitrogen Cycle.

of ammonia, and then by the oxidation of ammonia to nitric acid which generally unites with some mineral base present in the water. Thus free ammonia discovered by chemical analysis indicates that organic matter once present has started rapid decomposition. That which is not yet decomposed, whether dead or alive, is changed by chemicals to ammonia and is given as albuminoid ammonia, while ammonia which has been oxidised is recorded as "nitrites" and "nitrates" according as nitrious or nitric acid is formed. The nitrates indicate the final stage of decomposition of organic matter. The two forms of ammonias and nitrites are transition forms in this process

and show more or less recent pollution. When the ammonias are high and the nitrates low with a trace of nitrites, it indicates quite recent addition of nitrogenous matter to the water. When both ammonias and nitrites are low and nitrates high, it shows that polluting material has gained access to the water sometime in the past, but since then considerable purification has taken place.

The circulation of nitrogen from organic to inorganic state is termed *nitrification* ; this is effected by the nitrifying bacteria discovered by Winogradsky. W. T. Sedgwick represented the cycle of nitrification in a diagram as shewn in Fig. 11.

CLASSIFICATION OF THE PURITY OF SOURCES :—The purity of the sources of supply has been classified in different ways in different countries. In England it has been classified according to its palatability, while in America according to mortality.

In the sixth report of the Royal Commission on the River Pollution in England, the sources are classified as follows:—

Safe ..	1. Spring water. 2. Deep well water. 3. Mountain river, or lake water.	Very Palatable.
Suspicious ..	4. Stored rain water. 5. Surface water from cultivated land.	Moderately Palatable.
Dangerous ..	6. Low land river water. 7. Shallow well water.	Palatable.

J. H. Fuertes collected data from large number of European and American cities and classified the sources of supply according to death rates from typhoid fever.

Class A.	Mountain springs beyond zone of population.
,, B.	Water supply purified by slow sand filtration.
,, C.	Pure ground water supplies.
,, D.	Surface water with large impounding reservoirs and legal provision against pollution.
,, E.	Massive rivers purified by natural process.

Class F. Large inland lakes more or less subject to pollution.

„ G. All rivers and public wells polluted with sewage or other infectious matter.

These classifications may serve as an approximate guide to the selection, but the ultimate choice must rest on the reports of analysis of different sources.

METHOD OF COLLECTING SAMPLES :—The analyses must always be made by a public health chemist and for the purpose of analysis, representative samples from different sources of supply are required to be sent by the engineer designing the scheme. The results obtained in the analysis of water are frequently misinterpreted owing to the sample being not collected in the right way. In collecting samples for chemical and bacteriological examination the requirements to be fulfilled are not the same. The following are the requirements for chemical purposes as given by H. E. Stocks in his book on water analysis for sanitary purposes.

"Before taking a sample, the bottle is about half filled with the water taken direct from the source, shaken and emptied. The bottle may then be filled, the stopper put in and tied down. In taking a sample from a river or stream, the bottle is held mouth upwards below the surface of the water and if the stream is shallow, care should be taken not to disturb any deposit or mud at the bottom. At the time the sample is taken, certain particulars should be noted, for instance, number of sample, date, source of water (i.e. whether it is from well, spring, deep well etc.) nature of surroundings (i.e. whether any ashpits, cesspools, open drains or manure heaps are in close proximity). In the case of a well or spring the nature of the soil, subsoil, or rock may be noted, whether compact or porous, diameter and depth of the well, and in the case of a river or stream the distance from the source ; also the position (i.e. whether taken from the side or centre, from the surface or at what depth below the surface).

Any other information that may be useful in coming to a decision as to whether there is anything in the surroundings

that might now or in future render the water impure or unfit for drinking purposes should also be given.

A distinctive number corresponding with that in the note book is placed on each bottle at the time the sample is taken, this is very essential and saves a lot of trouble."

Dr. A. B. Griffiths describes in his Manual of Bacteriology the way in which a sample for bacteriological examination should be colected, viz :—

"To collect samples of water, accurately stoppered bottles (70 c.c. capacity) are used. These must be perfectly clean, and rinsed out with distilled water. Each bottle is put into a small tin canister, and the canisters (containing the bottles) are heated in a sterilizer to about 180° C. for at least three hours. The bottles thus sterilised can be easily transported without suffering contamination by dust to the place where the sample is to be collected. In collecting the sample of water the outside of the bottle should be rinsed in the water before removing the stopper ; and when the bottle is opened, the water is at once allowed to enter and fill the bottle to the extent of four-fifths, the stopper being immediately replaced and tightly screwed in so that exposure to the air is reduced to a minimum. The bottle is replaced in the tin canister, and the lid closed. After collection, the sample of water should be examined as early as possible. If the water has to be transmitted a considerable distance, occupying several days in transit, Dr. P. Mignel recommends the use of a "Glacier", or box in which the bottle is surrounded with ice".

The sanitary analysis and the collection of samples of water in this province is usually done by the Public Health Laboratory for public bodies.

ANALYSIS REPORT :—The analysis report is generally given in the following form :—

Total solids	...	...	...
Temporary hardness		...	...
Permanent hardness		...	...
Total hardness	...	...	...

Chlorine	...	...	...
Free ammonia N.		...	...
Albuminoid Ammonia N.		...	...
Oxygen absorbed		...	...
Nitrates	...	...	...
Nitrites	...	...	...
Sulphates	...	...	...
Phosphates	...	...	...
Iron and poisonous metals		...	...

Each of these substances present in water is estimated in part per 100,000 and its presence is usually interpretted by sanitarians as follows :—

TOTAL SOLIDS—The United States Geological Survey classifies water with less than 15 parts of total solids per 100,000 as low in mineral content, 15 to 50 parts as moderate and with 50 to 200 parts as high. Waters of low and moderate mineral content can be accepted for domestic purposes, and the highest quality of water should not contain more than 30 parts per 100,000.

SOLIDS IN SOLUTION—These are variable in quantity. It is not possible to lay down any hard and fast rule as to the significance of solids in solution in water ; hard water and water containing a large proportion of common salt may contain large quantity of solid matter. As already stated very soft waters are not the best from a dietic point of view, though they may be desirable for household and other purposes.

Rain water may normally contain 2 parts, while upland water 2·5 to 10 and deep well 1·5 to 60 or more solids in solution. The solid matter present in the Hooghly river water varies in different parts of the year and is also different in different places. It increases more and more as it approaches towards the Bay of Bengal. The solid matter present in the river water during the rains is nearly two and half times the quantity present during dry months. The rivers in Eastern

Bengal contain a less amount of solid matter than that present in the Hooghly water, although the nature of variation is almost the same.

HARDNESS—The following may be regarded as normal for water from the sources named ; the total hardness in degrees and the proportion of temporary and permanent hardness vary within wide limits.

TABLE 6

Source of Water.	Temporary.	Permanent.	Total.
Rain water	0·0	0·3	0·3
Upland surface water	1·4	3·6	5·0
Spring water	11·0	7·0	18·0
Shallow well water ..	12·0	22·0	20·0
Deepwell water ..	18·0	8·0	40·0

From health point of view there is nothing to choose between hard and soft water. In districts where very soft water is used, children are said to suffer from rickets owing to poor formation of bone. A moderately hard water, say, one of 10 degrees of hardness, is probably most suitable for domestic purposes. Water less than 6° is considered soft and above 14° hard, while water having hardness of more than 20° is considered very hard.

The total hardness of river water in this province varies very widely in different seasons. For the rivers in the western districts of the province the total figure varies from 5 per 100,000 during the rains to about 27 or 28 parts during the winters, while for the rivers in the eastern districts the total hardness varies from 5 parts per 100,000 during the rains to about 15 parts per 100,000 during the dry season. For surface water stored in tanks the hardness varies from 8 to 16 parts per 100,000 and the seasonal variation is not so marked.

CHLORINE :—The following may be taken as the average amount of normal chlorine present in various sources of supply.

TABLE 7

Rain water	...	0·2 part per 100,000.
Spring water	...	2·5 ,, ,, ,,
Upland surface water	...	1·2 ,, ,, ,,
Deep well water	...	5·0 ,, ,, ,,

In Bengal the normal chlorine quantity in rivers and surface water roughly varies from 0·6 to 1·2 excepting in the water from that portion of the country within the tidal zone.

As already said, the presence of this substance at one time was regarded as evidence of sewage pollution, but this view has since been discarded, as there are some supplies of exceptionally pure water containing considerable portion of the salt.

FREE AMMONIA :—A good sample of any class of water should not contain more than 0·005 per 100,0000 free ammonia ; a larger quantity is usually regarded with suspicion, as the presence of free and albuminoid ammonias indicates a rather recent pollution with organic impurities which has not had sufficient time for nitrification. The presence of free ammonia however is not a positive indication of danger to health. The amount present in the sample must be read in conjunction with those of albuminoid ammonia and chlorine present.

The following are the average amounts present in different sources of supply.

TABLE 8

Rain water	..	0·030 part per 100,000
Spring water	..	0·001 to 0·010 ,, ,, ,,
Upland surface water		0·002 to 0·008 ,, ,, ,,
Shallow Wells	..	2·000 to 2·500 ,, ,, ,,
Deep well ..	..	0·010 to 0·100 ,, ,, ,,
Lowland rivers	..	0·005 to 0·010 ,, ,, ,,

ALBUMINOID AMMONIA :—The term albuminoid ammonia was applied by Wanklyn to the ammonia produced by treating water containing nitrogenous organic matter such as protied etc. with strongly alkaline solution of potassium permanganate. It is some times called organic ammonia.

When it is present in quantities less than 0·002 parts per 100,000 the sample may be considered pure, but when a quantity larger than 0·10 per 100,000 is present, it generally indicates animal pollution and must be regarded with suspicion.

OXYGEN ABSORBED:—As a guide to the quality of various waters the following figures as given by Prof. Frankland and Dr. Tidy may be taken; the final conclusions, however, are liable to be modified by consideration from other analytical data.

TABLE 9

	Upland water. per 100,000	Other water. per 100,000
Water of great organic purity ...	0·01	0·05
Water of medium purity ...	0·10 to 0·3	0·05 to 0·15
Water of doubtful purity ...	0·30 to 0·4	0·15 to 2·00
Impure water	0·40	0·20

NITRATES AND NITRITES:—These are derived from oxidation of organic matter of animal origin and consequently indicate animal pollution at some past period. Owing to the fact that nitrates are final products of decomposition, no definite standpoint of purity can be fixed. Dr. Thresh reports that there are villages that have been using water containing 2 to 5 parts of nitric nitrogen per 100,000 but have yet been quite free from any water-borne disease, while serious epidemics of typhoid fever have occurred in places where the water supply contained only a small quantity of nitrates.

The limit usually given to mark off the non-suspicious water is from 0·2 to 0·4 parts per 100,000.

Dr. Frankland gives the following standard for the contents of organic elements of nitrogen and carbon in water.

TABLE 10

	Upland sources per 100,000.	Other sources per 100,000.
Water of great purity ...	up to 0·2	0·1
Water of medium purity ...	,, ,, 0·2 to 0·4	0·1 to 0·2
Water of doubtful purity ...	,, ,, 0·4 to 0·6	0·2 to 0·4
Impure water	above 0·6	above 0·4

The following substances, when present in more than the quantity mentioned against them, are considered deleterious to health :—

TABLE II

Lead ...	...	·01 part	per	100,000
Copper ...	...	·02 ,,	,,	,,
Zinc ...	...	·50 ,,	,,	,,
Sulphate ...	...	25·00 ,,	,,	,,
Magnesium	...	10·00 ,,	,,	,,
Iron ...	...	·03 ,,	,,	,,
Chlorides	...	25·00 ,,	,,	,,
Carbon Dioxide	...	1·00 ,,	,,	,,
Total Solids	...	30·00 ,,	,,	,,

The pH value should be less than 7, and in case of water with alkalinity below 5 parts per 100,000, pH value should exceed 8.

Bacteriological Examination :—In bacteriological examination no attempt is made to detect the presence of pathogenic germs in various samples of water. The standard test in this country includes the determination of *"total count"* and the presence of *B-Coli* per cubic centimeter of the sample. The significance of the presence of *B-Coli* has been a subject of much discussion. Originally it was found to be present in the contents of human intestine and was taken to be the characteristics of foecal pollution of men, but now it is generally recognised that it comes from the intestinal tract of worm blooded animals or birds. Its presence indicates that excreta of men or animals has obtained access into the water and has not yet been destroyed. This bacteria is more tenacious in life and survives longer in stored water than any pathogenic germs, which are more sensitive and short-lived. Hence, absence of *B-Coli* may be taken to represent absence of pathogenic germs. This bacteria has the further advantage that it lends itself more easily to laboratory methods, and also their presence can be readily determined.

In America another kind of bacteria known as *"Bacteria Aerogenes"* is gradually coming to be the standard of Bacteriological test in addition to that of *B-Coli,* owing to *Bacteria Aerogenes* possessing more resistance to the action of elements and also to the relative increase of growth over *B-Coli.*

In this country the following interpretation is usually put on the bacteriological examination of water:—

TABLE 12

Total Count of Bacteria per C.C.	
0— 100	Good potable water.
100— 500	Suspicious water.
500—1000	Bad water.
B-Coli, when present in 20 C.C. or above	Safe water.
,, when present in 5 C.C or above	Suspicious water.
,, when present in 1 C.C.	Dangerous water.

CHARACTERISTICS OF WATER FOR HUMAN CONSUMPTION—The following are considered to be most desirable characteristics of water for human consumption:—

The water should be—

(a) *Moderately soft*:—More than 30 parts per 100,000 in hardness is considered unsuitable.

(b) *Free from smell or taste*:—When heated to 37°C.

(c) *Cool and Transparent*:—Clarity of the water should be such that a platinum wire 1/25″ inch diameter shall be discernable 6 ft. below the surface in normal midday light.

(d) Should contain large amount of dissolved oxygen.

(e) Should contain sufficient free carbonic acid to make it sparkling and refreshing.

(f) It may contain the minimum quantity of organic matter and a trace of ammonia and no nitrites.

(g) It should be free from iron and should not contain more than normal chlorine.

(h) It must have no action on lead.

(i) Total bacteria count should be the lowest specified.

CHAPTER IV.

RAINFALL AND THE PORTION AVAILABLE FOR HUMAN SERVICE.

Rain being the origin of all sources of supply, a preliminary knowledge of its formation, character and variation is essential in dealing with the volume and permanency of a particular source.

Formation of Rain :—As is well known, the main physical processes involved in the formation of rain are evaporation and condensation. Solar heat is the chief evaporating agent. It transforms the water from the free surfaces such as oceans, rivers, lakes, moisture present in the vegetation, wet ground &c. into aqueous vapour, which finds its way into the atmosphere. When the atmosphere containing the aqueous vapour is cooled sufficiently, the latter condenses into minute small drops, which are held in suspension in the air by the force of the vertical currents in the atmosphere. A very large number of such drops held together appears as a cloud. If the process of condensation is kept up, the particles grow in size and overcoming the resistance of the vertical currents of air in which they are suspended, they fall down as rain. The condition favourable for inducing condensation in a cooling mass of air, have been studied in great detail in recent years, but need not be gone into here. It may however be useful to refer to the modes of cooling. This can be brought about by upward convection ; mixing of air masses of differing degrees of saturation and radiation. It is now recognised that without the first of these processes, any large scale and fairly continuous rainfall is not possible. Vertical convection in the atmosphere is very largely due to insolation and is also brought about by orographic features and meteorological conditions such as the juxtaposition and under cutting of warm moist air by cool dry air, and so on.

Indian Rainfall :—The moisture which is annually precipitated as rain in India is mainly brought in by the Monsoons or Seasonal winds. The principal rainfall season is that associated with the South-West Monsoon which prevails over Northern and Central India from June to October. This monsoon is part of a great system of air current of the Indian ocean which affects Africa, Indo-China, and Malaya as well. In winter the South-West Monsoon system is reversed. A dry northerly or north-westerly current sets in in Northern India, and flows across the Peninsula. This is known as the North-East Monsoon. The period of its activity forms the principal rainy season in the Coromandal Coast. Besides the monsoons, cyclonic storms originating in the seas and moving across the country and those moving across Persia into North-West India also contribute to the rainfall, particularly to falls of heavy intensities associated with the former class of storms. Finally, thunderstorms, as for instance in Bengal and Assam in Spring and Summer, are also responsible for intermittent showers.

Monsoon :—The main source of Indian rainfall, viz., the South-West Monsoon, is primarily due to the marked difference in temperature between the Indian continent and the adjoining oceans. Atmospheric pressure falls over the heated land area and the air over the comparatively cooler sea area therefore flows towards the land. This air current is laden with moisture owing to its passage over the sea. The course of the current and the rainfall that it produces are determined by several factors of which the configuration of the country is an important one. The Monsoon splits up into two branches below the latitude of Ceylon. One branch travels across the Arabian Sea, strikes the west coast of the Peninsula and passes through Gujrat, Central India to North-West India. The other branch moves over the Bay of Bengal, meets the Burma coast and after being deflected by the Arakan hills flows as a South-Easterly current over Bengal, Behar & Orissa and United Provinces. This demarcation of the field of activity of the two currents over Northern India is subject to variations according to the relative

strength of the currents and the movement of storms and depressions.

The Arabian Sea current meets the Western Ghats almost perpendicularly and is forced to ascend over the hills. This upward movement results in condensation and rainfall along the windward face of the Ghats and in the narrow coastal strip of land running parallel to the Ghats and West of it from the Kankan to the Cape Comorin. The region is one of abundant rainfall where the annual average is above 100″ inches and the average of the monsoon season alone is above 75″ inches. The falls are particularly on and near the summits of the Ghats. Some of these localities are believed to receive a record amount, second only, if not equal, to that of Cherapunji in Assam. The monsoon is thus deprived of a large part of its moisture while crossing the Ghats. Rainfall therefore begins to decrease abruptly on the Eastern slopes and becomes poor in the arid plateau of the Deccan. That part of the Peninsula which lies east of the Ghats gets only occasional falls in contrast to the heavy rain falling at times continuously for days together on the other side. The Bay Monsoon is subject to more or less similar orographic influence on the Burma Coast. Rainfall is heavy along the coast from Arakan down to Tennasserim and decreases in land where a dry zone exists over Central Burma. The monsoon current on crossing the head of the Bay of Bengal blows directly against the Khasi hills, where rainfall is again excessively heavy. As is well known, Cherapunji on the Khasi hills has an annual average of 426″ inches, which is the greatest amount recorded in any part of the world. It has already been pointed out that the effect of the Eastern Himalayas and their Southward continuation down to the Arakan Yoma ranges is to deflect the Bay Monsoon into a South-Easterly current over Bengal and make it go up the Gangetic plain as a South-Easterly and Easterly stream. This branch of the Arabian Sea current which enters North Western India and the United Provinces are bordered by the great barrier of the Himalayas to the North, the Frontier Chain on the North Western and the Southern continuation of the Eastern

Himalayas on the East. All the moist air flowing in is thus trapped on all sides and therefore forced to rise and precipitate its moisture. This, in general terms, is the way in which Assam, Bengal, the Gangetic Plain and the Central parts of the country derive their general and widespread rainfall. North West India comprising the Punjab, Rajputana, Sind and the North West Frontier Provinces receives poor rainfall compared to the rest of the Indian Monsoon area, Sind and its neighbourhood being the driest zone with an annual rainfall of less than 10″ inches.

The monsoon is not a steady, continuous flow but is pulsatory in character. An advance of the current takes place, its activity lasts for sometime, then a break occurs and again another advance follows. Normally the monsoon breaks on the Arabian Sea coast in the first week of June and in Bengal in the second week.

Besides the winds and the configuration of the country in relation to their direction, there are other general factors like the altitude and the presence of forests which also affect rainfall. Although forests are believed to make conditions favourable for rainfall, this has not been established by the investigations so far made. The question is still a subject of controversy.

Measurements of rainfall :—Measurements of rainfall are usually recorded in inches of fall of rain, hail or snow when melted upon a horizontal surface free from drifts. The instrument employed in determining the amount of rainfall in this country is Symon's Rain Gauge as shewn in Fig. 12.

The gauge consists of a copper cylinder provided with a funnel of the same metal to receive the rain and prevent evaporation. The tube of the funnel terminates into a glass cylinder which retains the rain water until the observer has measured the depth of water in a graduated glass measure designed for the purpose. The funnel is usually 5 inches in diameter in this country and the top of the gauge should stand at a height of 12 inches clear of the ground. The side should be on open ground

with plenty of open space and at a distance of not less than one and half times the height measured from plantations or buildings. The height at which the rain gauge is fixed is important. Experience has shown that the higher the gauge is placed above

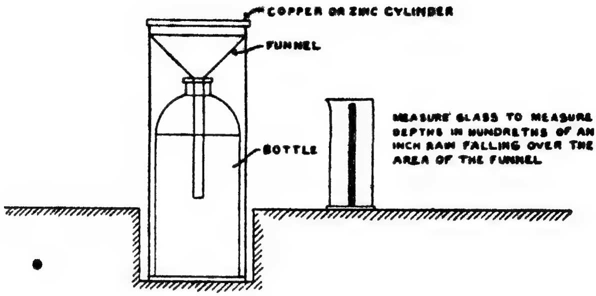

Fig. 12—Rain gauge.

the ground level, the smaller is the quantity of rainfall recorded owing to the eddies and currents that carry away drops that would otherwise fall into the mouth, while if it be placed too low, more rainfall than the actual will be recorded owing to the spattering into the glass water falling on the ground and the neighbouring objects. In Alipur Observatory, a gauge placed 50 ft. above ground level recorded only 87% of the actual rainfall during 7 years observation.

Rain-gauges are usually read once in 24 hours which is generally done at 8 A.M. everyday in this country.

In schemes of water supply or irrigation unqualified reliance on a single gauge close to the district may cause serious error in estimating yield. The number and selection of sites for fixing rain-gauges should receive careful attention. Sir John Benton, Inspector-General of Irrigation, gave the following least number necessary for the area mentioned against them for reasonable estimate of rainfall on the basin.

TABLE 13

Area in sq. mile	0 to 50	..	1 gauge.
,, ,, ,,	50 to 100	..	2 gauges
,, ,, ,,	100 to 200	..	3 ,,

For basins of small magnitude and for more accurate results there should be two rain gauges for every 1,000 acres. Dr. Deacon suggested that there should be at least 5 gauges in a hydrographic basin to obtain good results, three on the axis of the valley of which one should be near the junction of that axis with the boundary of the watershed and the two on the middle of sloping sides of the basin.

Rainfall in this province is usually published in the *Calcutta Gazette* every week and also in book form annually. There are about 250 recording stations in this presidency, of which records over 35 years are available for 61 stations.

An isohytal map shewing the mean annual rainfall on different parts of India is given in annexed page ; a study of this and the reports periodically published shows that the total annual rainfall is seldom the same for two years at the same place, or at any two places the same year ; and the variation seems to follow no fixed law. During a dry season at one place, another place scarcely 100 miles away, may be visited by a wet one. For this reason the rainfall of a locality is taken to be the mean annual rainfall observed over a period which is sufficiently long to produce a fairly constant mean. Sir Alexander Binnie in a paper read before the Institute of Civil Engineers in England stated that observation over a period of 30 or 35 years are necessary to ascertain the mean annual rainfall of a district. This was deduced from the records of 42 stations all over the world in which the rainfall records for periods varying from 50 to 97 years were available, amongst which were included Bombay, Madras and Calcutta. Prof. Bruckner of Bern who has made careful investigation of the whole subject of climatic changes finds evidence of 35 years' periodicity in temperature and rainfall. Added to this W. J. S. Lockyer has called attention to the fact that there seems to be a periodicity of 35 years in solar activity. The Hindus however have a cycle of 60 years, the result, as supposed by western scientists, of centuries of observation each year bearing a name which indicates among other events likely to

happen the nature of agricultural season and so indirectly the rainfall. This is known as *Bahu-dhanya.*

Consequently 30 or 35 years is the shortest time that can be profitably considered in arriving at a mean annual rainfall of a place. The following table as prepared by Sir Alexander Binnie gives the deviation of the mean value of the annual rainfall during the whole of the record expressed in percentages of the mean fall for each period.

TABLE 14

Deviations from the mean value of the annual rain-fall during the whole period of the record, expressed as percentages of this mean fall, when the period considered is :—

Years	5	10	15	20	25	30	35
Maximum Positive deviation ..	23·20	14·90	9·20	5·60	7·30	5·20	4·50
Maximum Negative deviation ..	29·60	16·10	12·50	9·20	9·00	6·90	4·70
Average Positive deviation	15 35	8 08	3·87	2·47	2 56	2 17	1·73
Average Negative deviation .	14·52	8·37	5·64	4·08	2·94	2·36	1·86
Minimum Positive deviation ..	6·80	1 00	0·00	0·00	0·00	0·00	0·00
Minimum Negative	7·80	4·70	0·80	0·00	0 00	0·00	0·00
Average deviation ..	14·93	8·22	4·77	3·27	2·75	2·26	1·79

The mean annual rainfall over a long period can be determined for an area upon which the actual fall is recorded for a comparatively short period by assuming that the mean fall of the locality for the short period bears the same ratio to the mean fall of the locality for longer period as the mean fall of another locality close to it for the same short period to the meanfall for the same longer period. Thus—

If r = mean rainfall in the standard station for short period of n years.

R = mean rainfall for longer period of N years.

A = mean rainfall of new station for n years.

B = mean rainfall of the new station for N years to be found out.

then $B = \frac{R\ A}{r}$

It is also important to know the relation between the mean annual rainfall of the district and the annual average rainfall for the period of longest drought. In England two laws are generally accepted by engineers in calculating the minimum fall of a locality, *viz.* Glashier's law and Hawksley's law. According to Glashier's law, the average fall of three consecutive years yielding minimum fall may be considered as the average minimum fall, while according to Hawksley's law, from the average rainfall of twenty years is deducted one-sixth the average of the three years of minimum fall. The former law was adopted in the calculation of minimum rainfall for Topchanchi reservoir of the Jharia waterworks.

TABLE 15

Years of drought.	Per cent. of mean 25 years rainfall.
1	61
2	72
3	77
4	80
5	82
6	83·5

The above figures are based on an article by Dr. Deacon on water supply giving the relation between the average rainfall of different periods of driest consecutive years or years of drought and the mean average rainfall of 50 years.

From the report of irrigation commission of 1901-03 we find that periods of severe droughts in India usually vary from 1 to 6 years with the exception of the west of the Punjab and Hyderabad where the drought has extended over a period of 8 years or more. In Bengal the period of severe drought is never more than 3 years.

Before proceeding further, it is necessary to understand the various terms used in connection with the disposal of rainfall.

Rainfall means the mean annual fall in inches of rain, hail or snow observed over a period which is sufficiently long to produce a fairly constant value. The period of observation should not be less than 30 to 35 years.

The *evaporation* means the mean annual evaporation in inches of the depth of water evaporated from a free water surface.

The *absorption or percolation* is the depth in inches of rain water that annually penetrates into the earth or is absorbed by vegetation.

Run off or flow off is the portion of the rain fall that flows off a catchment basin to form streams, rivers or lakes. This is expressed in various ways either in inches of the depth of water over the catchment basin or a percentage of the rain falling over its surface and the maximum run off is sometimes expressed in cubic feet of water flowing off the catchment per second.

Evaporation and absorption :—Mr. Buckley in his Irrigation Pocket Book says: "It is a very difficult matter to ascertain the proportion in which the evaporation and absorption respectively play their parts. The proportion differs enormously according to the conditions. The part which each assumes varies greatly in most cases in different times of the year, according as the air is dry and saturated, according to the force and even the direction of the wind, according to the nature of the soil, and particularly according to the degree of saturation of the soil at the moment."

The amount to be deducted on account of evaporation and absorption is therefore a matter of experience based on the observations made so far in some places.

The facts given hereafter may serve as a guide in determining this uncertain factor in other places.

The following table gives the evaporation from a circular

pan 3 ft. in dia. floating in the Red Hill reservoir situated 8 miles from Madras:—

TABLE 16

Month. 1915.	Evaporation in inches.	Mean temp.	Percentage of Relative Humidity.	Rainfall.
January ..	3·37	76·3	80	5·96
February ..	4·35	78·5	77	0·53
March ..	6·30	81·8	77	0·00
April ..	7·60	84·7	76	0·06
May ..	9·00	80·6	65	0·21
June ..	6·60	88·2	67	1·45
July ..	5·85	84·7	74	7·78
August ..	7·35	85·4	72	1·76
September ..	8·05	83·8	76	3·31
October ..	2·30	83·5	78	2·64
November ..	—	79·1	85	19·55
December ..	1·40	76·3	74	1·02

In the above table the evaporation for November is not shewn as no observation was made owing to the instrument getting out of order, and the figure for December is for 10 days from 22nd to 31st. The temperature and humidity given in the above table are for Madras.

Another observation was made in Tansa reservoir, Bombay.

TABLE 17

	1890	*1891*
January ..	4·68	5·28
February ..	7·56	5·64
March ..	6·48	7·56
April ..	6·84	7·68
May ..	9·73	9·36

Sir Alex. Binnie made observations in Ambajhari tank at Nagpur and found that it averaged during the dry season at 1/5 inch. per day.

Major Cunningham found from the Ganges Canal, Roorkee, that the maximum evaporation did not exceed 0·37 inch in one

day and excluding the monsoon period it averaged 0·1 inch per day. The average for Bengal would certainly be less than that at Roorkee.

The following is extracted from a table prepared by Capt. Ward, R.E. for estimating the amount of monthly evaporation in a large tank from a given mean temperature and relative humidity,

TABLE 18

Mean temp.	65°	70°	75°	80°	85°	90°	93°
Degree of humidity Saturation = 100	Ft.	Ft.	Ft.	Ft.	Ft.	Ft.	Ft.
88	0·14	0·17	0·19	0·21	0·24	0·27	0·30
85	0·19	0·22	0·22	0·23	0·27	0·30	0·33
80	0·22	0·27	0·29	0·30	0·32	0·36	0·38
75	0·24	0·29	0·33	0·35	0·37	0·40	0·42
70	0·26	0·30	0·34	0·35	0·38	0·42	0·44
65	0·28	0·31	0·34	0·36	0·40	0·44	0·46
60	0·30	0·35	0·38	0·40	0·42	0·47	0·49
55	0·32	0·37	0·42	0·46	0·50	0·52	0·54
50	0·35	0·40	0·45	0·50	0·55	0·60	0·62
45	0·45	0·50	0·57	0·67	0·71	0·75	0·78
40	0·61	0·60	0·70	0·72	0·77	0·82	0·84

The temperature and humidity of different meteriological stations are published by Government.

The data regarding absorption are still more meagre and hardly applicable to the conditions prevailing in Bengal. A fairly safe rule for the plains of Bengal is to allow a depth of 5 feet over the average area of a tank or reservoir to cover the loss from evaporation and absorption combined.

Run off :—The total yield from a catchment area in inches of fall over the basin is equivalent to the total rainfall less loss in inches due to evaporation and absorption by soil and vegetable. As all these factors and the relation between them are very little understood and the data regarding them are very meagre and incomplete, the only course open to an engineer is to assume the run off to be a certain percentage of the

rainfall of the locality. The percentage is based upon experience, judgment and gauging of the existing stream draining another watershed in the neighbourhood. The amount of run off varies with the climate, character, formation and condition of the surface and also of the subsoil, the meteorological conditions before and after each fall, and also the geographical and topographical position of the watershed in reference to the existing tanks, marshes or other bodies of water. That the run off from two watersheds should be the same it would be necessary that they must be exactly similar in shape, in ground slope, in soil, in amount of forest and vegetation, in area of existing tanks in the movement and humidity of the atmosphere and lastly in the character, amount and distribution of rainfall of the locality. Since all these factors of no two watersheds can be similar, the run off from them must necessarily vary. The amount of run off increases with the amount of rainfall. It is greater in the wet season when the ground is more saturated and absorption is less. It varies greatly with the compactness of the soil and is very slight in sand. It is considerably influenced by the slope of the surface on which it falls. The diagram (Fig. 13) shows

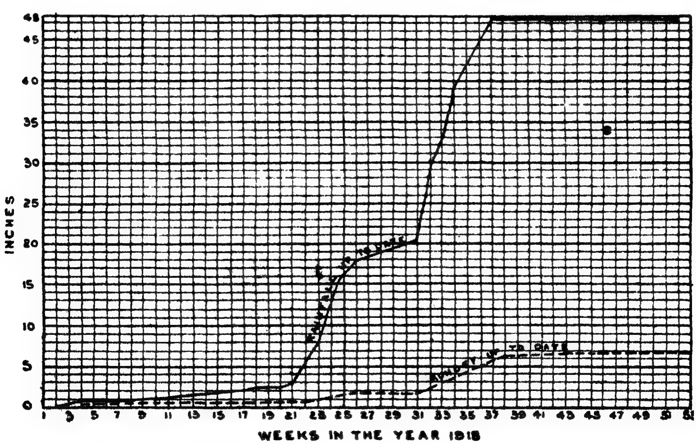

Fig. 13—Relation between rainfall and run off—
Jharria Watershed.

the aggregate rainfall and run off up to different weeks of the year 1918 of Topchanchi watershed. This diagram clearly shows how the run off from a particular watershed varies according to the intensity of rainfall and that the rate of run off does not increase immediately at the same ratio as the rainfall increases.

The Topchanchi watershed is between two ranges of hills having steep sides and being full of forest and, only on a fraction of this, rice is cultivated.

W. L. Strange in his book on "Indian Storage Reservoirs" gives the following table for estimating approximately the run off from different classes of watersheds :—

TABLE 19

Daily rainfall in inches.	Run off percentage and yield when the original state of the ground was—					
	Dry		Damp		Wet	
	Percentage.	Yield in inch.	Percentage.	Yield in inch.	Percentage.	Yield in inch.
0·25	...	...	...	...	12	0·03
0·50	...	...	10	0·05	14	0·07
0·75	...	...	12	0·09	17	0·13
1·00	5	0·05	14	0·14	20	0·20
1·25	6	0·08	16	0·20	23	0·29
1·50	7	0·11	19	0·29	26	0·39
1·75	8	0·14	22	0·39	30	0·53
2·00	10	0·20	25	0·50	34	0·68
2·50	15	0·38	32	0·80	43	1·08
3·00	20	0·60	40	1·20	55	1·65
4·00 & over	30-40	1·20	50-60	2·00	70-80	2·80

The following rules have been devised in the Madras Irrigation department for the classification of transition of one into other classes of catchment in the above table.

1. Transition from dry to damp requires a rainfall of

¼" 1 day before.
½" 3 days before.

1" 7 days before.
1½" 10 ,, ,,

2. Transition from dry to wet requires a rainfall of 2½ inches in the previous day.
3. Transition from damp to wet requires a rainfall of

⅓" 1 day before.
½" 2 days ,,
1" 3 ,, ,,
1½" 5 ,, ,,

4. Transition from wet to damp and from wet to dry will be when the falls are half and ¼th respectively as given in rule 3.

An investigation of the relation between rainfall and run off was made on a tract of land mainly composed of ricefields and having a small slope near Calcutta. The result of the investigation is given in a table in Buckley's Irrigation Works in India and Egypt which is extracted below:—

TABLE 20.

	During normal monthly fall.	When monthly fall is above normal.	Remarks.
June ..	5	10	End of hot weather.
July ..	10	20	
August ..	25	50	Monsoon established.
September ..	40	50	
October ..	40	50	End of monsoon.

A fairly safe rule for estimating minimum annual run off from a watershed on the plain of this Presidency and Behar & Orissa is to take it to be 35% of the average rainfall of three driest years during the monsoon months from June to October.

Maximum run off—Before leaving this subject it is necessary to refer to one matter in this connection which a waterworks engineer has often to deal with, *viz.* the flood discharge or the maximum run off from a catchment basin.

This is generally expressed in cubic feet of water flowing off the area in one second. The maximum discharge from a watershed is very important, since requisite provision for weirs and spillways for an artificial lake or reservoir entirely depends upon this discharge.

The maximum discharge from a catchment basin depends among other things on the following :—

(i) The degree and duration of rainfall.

(ii) The degree of staturation of the ground, while other conditions remain the same, is dependent upon the intensity and duration of rainfall.

(iii) The area and shape of the watershed.

(iv) The area, shape and topography of the basin.

(v) Storage both natural and artificial—depressions or lakes, tanks, etc. within the watershed area act as a flood moderator and reduce the maximum run off.

Maximum flood generally occurs when combination of many of the above circumstances occur.

Several empirical formulae have been devised to express the maximum rate of run off from a catchment basin. The following are a few of them :—

Col. Dickens	$CM^{\frac{3}{4}}$
Col. Ryves	$CM^{\frac{2}{3}}$
Fanning	$CM^{\frac{5}{6}}$

where C is a constant depending upon the conditions of the watershed and the intensity and duration of rainfall over it.

M is the area of the watershed in sq. miles.

The first two formulae are generally used in this country and the last in America.

General Mullins, R.E., sometimes Chief Engineer for Irrigation to the Government of Madras, laid down the following rules for obtaining value of C in Col. Dickens and in

Col. Ryves formulae. If "x" be the average of the maximum recorded rainfall in 24 hours over a watershed basin, then

C in Col. Dickens formulae	...	40 x
C in Col. Ryves formulae	...	48 x

This rule is intended for use in the Madras Presidency. The present writer compared the constants obtained from this rule with those for known flood discharges of some of the rivers in this Presidency and found them to be high. It is believed that between 30x and 36x will be the probable figure for Bengal.

The value of "C" in Col. Dickens formulae suitable for conditions prevailing in Bengal ranges from 300 to 850, the approximate value for Mahanady, Subarnarekha and Brahmani being about 500 and that of Damodar, Cossaye, and Baitarani about 830.

CHAPTER V.

VOLUME AND PERMANENCY.

Classification :—For the purpose of estimating the volume and permanency of a supply, the source may be grouped into two principal classes, *viz.* :—

1. Surface water sources.
2. Ground water sources.

Surface water Sources :—The supplies obtained from (1) a stream or river, (2) a natural lake or an artificial reservoir, (3) water collected from roof surface are all surface water in one form or another. The yields in the case of the first two are obtained from natural catchment basins, while in the case of the last from an artificial gathering ground. Lately water works have been constructed in some towns in Bengal which depend for their supply upon an artificial gathering ground the waters from which are collected in tanks excavated within the area.

When no storage in the form of an excavated tank, or impounding reservoir exists, the volume of continuous supply from any watershed area is limited to the minimum dry weather flow of the stream draining it. This cannot be determined from the rainfall ; it entirely depends on the capacity of the soil or rock to store water in the particular basin and to yield it to the stream by means of concentrated springs or diffused seepage. At the same time it must be remembered that the total annual yield from a river basin can never exceed the quantity of water precipitated in the form of rain, hail or snow.

Measurement of flow :—The flow of a river or stream is generally gauged by either of the following ways :—

1. By means of rectangular or V notch or a compound weir.
2. By taking cross section of the stream and velocity observation.

GAUGING BY NOTCH OR WEIR—The first method is usually adopted in the case of small rivers, streams or springs. In this sort of gauging it is necessary to dam up the stream of water and form a considerable pond in which the water may expand itself and flow out quietly as nearly still water as possible. If this precaution is not taken the velocity of the water before arriving at the gauge would have to be measured at the time of every observation and this would admit numerous possibilities of error.

These gauges being of a temporary nature may be made of planks, the joints being grooved and tongued and the whole bolted together and stiffened by battens. Fig. 14 shews the

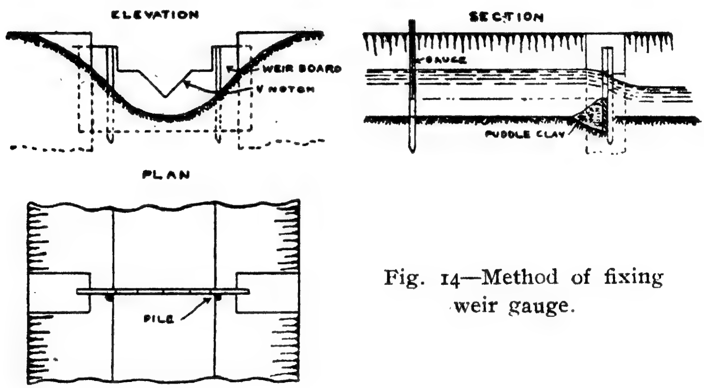

Fig. 14—Method of fixing weir gauge.

sketch of a V notch fixed in the bed of a stream. A stream should be gauged at least once a day, but that is not sufficient for accurate results, for within 24 hours the depth of flow of water over the gauge may vary very greatly. Observations are therefore necessary as frequently as can be arranged ; these observations are to be supplemented by notes of the beginning and end of rains influencing the flow of the stream. With reference to the dimension of the gauge, regard must be had to the catchment area of the basin and to the probable variation in the rate of flow of water. The gauge must allow the floods to pass over it, at the same time part of it must be small enough in width to gauge the dry weather flow ; the one quantity

may be a hundred times more than the other. It is in such cases that a compound weir is reliable.

The first thing to be done in fixing a gauge weir across a stream is to divert the water from its normal channel and excavate a trench across the bed of the stream and well into the banks on each side to a width of not less than 7 or 8 feet and to a depth of not less than 5 ft. below the bed of the stream according to the nature of the soil—the trench to be filled in with puddle clay. When a depth of 3 ft. has been formed at the bottom, the gauge should be set perfectly level upon it, and the filling of the trench with puddled clay on the upper side completely up to the bed of the river. Drive two stout stakes on the down stream side of the guage to keep it in an upright position and ram puddle clay all round the board on the upside to ensure perfect water-tightness. Having made the dam watertight, test the level of the V or rectangular notch, the top of the notch must be truly horizontal. It is necessary to see that there is a free overfall from the notch, otherwise there is a tendency for inaccuracy in readings.

As the water approaches the weir, the surface begins to incline towards the weir until the notch board is reached, where it forms a parabolic curve in falling over. To calculate the quantity of water passing over the weir, it is necessary to measure the actual depth of still water behind the weir, the top of which should be at the same level as the lip of the weir ; therefore a gauge peg should be fixed some 4 or 5 feet away from it and at an easily accessible place near the bank on the upstream side. The depth may be measured either by Boyden's hook gauge (Fig. 15) or by making the gauge peg of a properly graduated piece of wood or metal and fixing its zero level coincident with the sill of the weir or notch. Of these the more

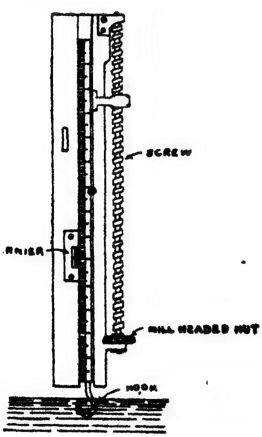

Fig. 15—Hook gauge.

accurate way of measuring the exact water level is by Boyden's hook gauge. It is made up of a fixed frame, a sliding scale and vernier, and a hook fixed at the bottom. The vernier is fixed to the body of the frame and the scale moves up and down with the hook fixed at its bottom. The scale is lowered or raised by means of fix-pitched millheaded nut at end of the screw on which the scale is attached. The hook is first brought below water level and it is very cautiously raised until the end of the hook just touches the water surface. This is exhibited by the capillary elevation of water surface over the point of the hook. The scale is divided into inches and tenths, and the vernier gives reading up to 100th of an inch. For the purpose of measuring continuously the depth of water over a weir, there are several instruments in the market. Fig. 16 shows one usually used in such cases. It is provided with a strong 8 day pendulum clock which works the drum on which the depths of water at different periods of the day are indicated by means of a pen-carriage worked by a float placed on the upstream side of the weir. The travel of the pen on the drum is usually 6 to 12 inches and can be reduced to any suitable ratio required. The recorder can be fitted with a clock and drum to give a daily or weekly diagrams as required. Another instrument of the same type which can be used as recorder is designed so as to give the rate of discharge over the "V" notch, rectangular notch or compound notch, and at the same time integrate and give the total quantity that is passed over in any particular period of observation. The Glenfield Recorder is an instrument of this description and is illustrated under Fig. 16.

Having thus set the apparatus in a true and satisfactory manner, it remains to calculate the flow by means of a reliable formula. The discharge for various depths may be calculated from the Chart No. 1 which is based on a table given by Mr. M. R. Collins in the bulletin No. 3 of the Transval Department of Irrigation, and is considered to be reliable.

Gauging by Velocity Observation—A preliminary difficulty arises in observation of velocities in any point in a stream or river owing to the velocity rapidly varying and the

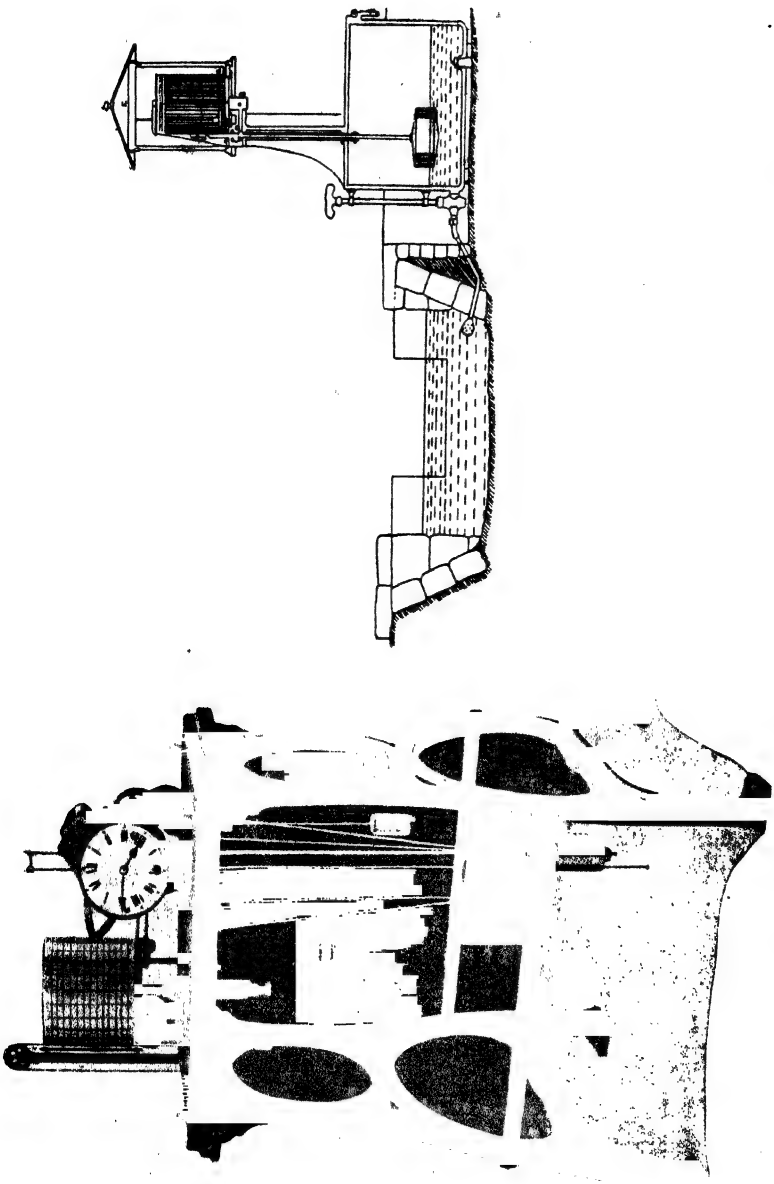

Fig. 16—Arrangement for fixing Continuous recorder and the Gauge Weir.

motion not being steady. If, however, an average of several velocities at the same point is taken, it is found to be fairly constant. The fluctuation is probably due to the eddying motion superposed on the general motion of the stream, but these fluctuations produce effects which disappear in the mean of a series of observations and may therefore be neglected in calculating the volume of flow.

In the next place, it has been observed in numerous gaugings of rivers that the maximum velocity in a vertical plane is not at the surface but at some distance below it. In many observations, this depth at the centre of the stream has been found to vary from 0 to 0·3 of the depth.

With regard to the finding of mean velocity on a vertical plane from two or more velocity observations, the results as obtained by Col. A. J. C. Cunningham in gaugings on the Ganges Canal and expressed in the formula below may be found useful.

Let, V_0 = surface velocity

V_m = mean velocity

Then $V_m = \frac{1}{4}[(V_0 + 3V(\frac{2}{3}d))]$

where the $V(\frac{2}{3}d)$ = the velocity taken at ⅔ depth

and when they are taken at $\frac{1}{4}$, $\frac{1}{2}$ and $\frac{3}{4}$ of the depths, then

$$V_m = \frac{1}{3}[2\,V(\tfrac{1}{4}d) - V(\tfrac{1}{2}d) + 2\,V(\tfrac{3}{4}d)]$$

Bazin however found that the mean velocity for the whole section of a channel fairly regular in section varies from 0·7 to 0·85 of the greatest surface velocity. For channels not widely different from those experimented, the following formula given by Bazin may be taken as a good approximation:—

$$V_m = V_0 - 25{\cdot}4\sqrt{mi}$$

Where m = hydraulic mean depth

i = sine of slope of stream surface

A portion of the stream, where the channel is fairly uniform for a length of not less than 200 ft. and in the course of which there are no eddies, is to be selected for observation. Three or four cross sections are to be taken at equal distances if possible. When there is water in the channel, wire may be stretched level

across it along which soundings may be taken with a rod or plummet. The mean velocity of a stream may be measured by hollow velocity rods and immersed at various distances from the banks which are allowed to float from one cross section to another and the time of passing each cross section being noted with a stop watch. The cross sections should be divided into segments as nearly equal as possible and the rod should pass if possible from the centre of one segment of one cross section to the centre of the corresponding segment of the next cross section. The time taken for a rod to pass through the run will give the mean velocity of the particular segment.

Mean velocity rods as shewn in Fig. 17 are tin cylinders about 1 inch in diameter and the lower portion of which is formed of such a length of iron rod that only a short length varying according to the depth of water in the channel is left to project out of the surface of the stream. The fine adjustment may be made by small shot and the piece of rod and the ends permanently sealed. The rods should be painted black with a ring of red paint to mark the water line. The following table gives the different height of projections of rods above water surface according to various depth of water in the channel :—

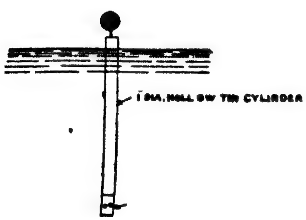

Fig. 17.

Table 21

Depth of channel.	Rod projection.
Above 6 ft.	3 inches.
Between 3′ & 6′	2 ,,
,, 1′ & 3′	1 ,,
Below 1′	½ ,,

The proper length immersed of the rod is from 0·93 to 0·95 of the total depth of water in the vertical.

The following two other methods were used in Roorkee experiments in observing velocities of streams.

1. A pinc wooden disc 3 inches in diameter and ¾ inches in thickness is connected by a fine brass wire (30 B.W.G.) to a special ball 3 inches diameter of some heavy wood. The ball should be boiled in oil to eliminate its power of absorption and adjusted in weight by boring out a portion of its mass from inside or by putting lead shots in a hole made for the purpose, with a view to keep the attached float flush with the water surface. The time taken by the float to pass through the run gives the velocity through the vertical at the depth to which the ball is immersed and the mean velocity may be obtained by the formula given in page 58.

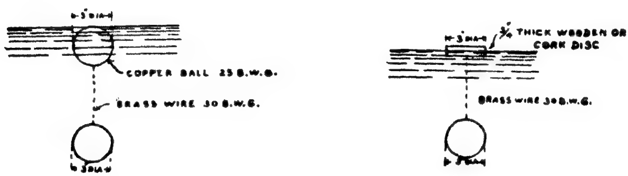

Fig. 18—Twin and sub-surface float.

2. Fig. 18 shows two equal balls connected by a wire of the same gauge mentioned above, the lower one being so loaded that the top of the upper one just touches the surface of the stream. The motion of the twin must be nearly the mean of the surface velocity and the velocity at which lower float moves. Surface velocities were measured by Col. Cunningham at Roorkee by means of 3 inches diameter and 1 inch thick deal wood discs. When Vs=surface velocity Vd=velocity at depth, then velocity of the twin float V will be equal to ½ (Vs+Vd). Both surface velocity and velocity of twin float being known by observation, Vd can be easily found. For the calculation of mean velocity of the section, same method as in the case of

surface float is to be adopted. Col. Cunningham found the twin floats to be more satisfactory than the former, but the influence of connection increased with the depth and also there is the uncertainty of the lower float keeping its depth owing to the eddies of the water tossing it about.

Determination of Storage—When the natural continuous supply, as determined by gauging, is found to be smaller than the consumption, storage will have to be provided to maintain a balance between the fluctuation of supply and demand. The method of calculation, usually adopted in cases of all supplies either from a river or stream, or a natural catchment area or an artificial gathering ground, is the same. An example is given below in case of a supply from an artificial collecting surface.

In case of a supply from a natural watershed area, after obtaining a measurement of the drainage area and forming a decision as to the probable average minimum and maximum yields from the area and the consumption to be provided for, a calculation as to the sufficiency of the source can be made. To determine the watershed area it is generally sufficient to mark off the watershed lines on the topographical maps which can be obtained from the map department of the Survey of India office, and to measure the enclosed drainage area above the line of interception by means of squares or a planimeter. When however the watershed lines are not well defined, a contour survey of the ground is necessary to mark off the water parting lines. Having thus determined the area of the water shed, the lowest perennial flow of the stream draining it and approximating in the manner previously explained, the amount of run off, absorption and evaporation, the amount of storage necessary may be estimated.

Rainfall in this country not being uniform throughout the year and limited generally to the south west monsoon period except in Madras Presidency, the yield from any collecting basin must necessarily be variable, and storage is necessary to equalise the flow of water over a long period of drought. If the minimum daily yield from a catchment area in the form of a

stream or spring is equal to more than the maximum daily supply, no storage will be necessary; if the minimum annual yield is equal to or greater than the maximum annual consumption for which the waterworks is designed, then enough storage to carry over the dry season of one year of drought is to be provided for; but if the mean annual rainfall is nearly or quite equal to the mean yield required of the watershed, then all the surplus from the years of greatest rainfall must be stored to meet deficiency during the periods of drought.

In making calculation for storage, evaporation from the reservoir must be considered; it is proportional to the water surface in the reservoir and for preliminary calculation an area is to be assumed.

There will be some loss from a reservoir due to absorption. If no other data are available, evaporation and absorption may be taken to be 10% of the consumption. This may be much too little in some cases; no definite proportion can be laid down for it in proportion to consumption. The consumption should be increased or the yield decreased according to judgment.

The following is the most commonly adopted method of determining the capacity of storage. Any probable quantity is first assumed for the capacity and the reservoir is supposed to be full at the beginning of the period of drought. By simple addition of supply and subtraction of consumption during several months or days according to the nature of the watershed, the calculation is continued. Should there be any negative or minus result, the capacity is increased and the calculation is repeated. For example, take the case of a supply to a family consisting of 16 members living at Krishnagar. From rainfall collected from a roof surface of 2500 sft. and the water being required for domestic purposes only, about 100 gallons of water per day will be necessary to meet the requirements of 16 persons for domestic purposes only. The whole family therefore will require about 36,500 gallons of water in one year.

The mean annual rainfall at Krishnagar is 58·40, and if we assume that 36 inches of this fall can be collected from the roof

TABLE 22

Statement of Storage Calculations.

Months.	Amount of storage on the 1st of the months in Galls.	Amount of rainfall in inches.	Nos. of rainy days.	Amount in inches lost in first washing.	Nett rainfall in inches.	Amount available for storage in Galls.	Consumption during the month in Galls.	Surplus stored in Galls.	Deficiency drawn from storage in Galls.	Amount left over at the end of the month.
January ..	0·0	·44	2	·08	·36	468	3,100	......	2,632	−2,632
February ..	−2,632	1·20	2	·08	1·12	1,456	2,800	......	1,344	−3,976
March ..	−3,976	1·82	2	·08	1·74	2,262	3,100	......	838	−4,814
April ..	−4,814	2·55	6	·24	2·31	3,003	3,000	3	......	−4,811
May ..	−4,811	7·00	5	·20	6·80	8,840	3,100	5,740	......	929
June ..	929	10·66	10	·40	10·26	13,338	3,000	10,338	......	11,267
July ..	11,367	11·08	16	·64	10·44	13,572	3,100	10,472	......	21,739
August ..	21,839	10·17	11	·44	9·73	12,649	3,100	9,549	......	31,388
September ..	31,387	8·03	10	·40	7·63	9,919	3,000	6,919	......	38,307
October ..	38,306	4·38	7	·28	4·10	5,330	3,100	2,230	......	40,537
November ..	40,536	·88	2	·08	·80	1,040	3,000	......	1,960	37,577
December ..	37,576	·09	1	·04	·05	65	3,100	......	3,035	34,542

January	February	March	April	May	June	July	August	September	October	November	December
6,005	3,375	2,031	1,193	1,196	6,936	12,000	12,000	12,000	12,000	12,000	9,040

surface in the driest year, then the annual yield from the 2500 sft. of the roof surface is equal to $\frac{2,500 \times 36 \times 6 \cdot 25}{12}$ galls. = 46,875 gallons. Therefore, the roof surface and the rainfall are sufficient to meet the annual requirement of the family. The rainfall is however not uniform throughout the year, consequently storage will be necessary to equalise the supply over periods of drought.

For the purpose of calculation in the present case, the reservoir in which the water is to be stored is assumed empty on the 1st Jan. and that the run off from the roof surface to be 100% of the rainfall. It will appear from the statement that an allowance at 2 gallons per 100 sft. has been made for allowing the first washing of the rain to run to waste. The storage condition of the reservoir from month to month is given in the statement,* from which it will appear that at least 4811 gallons of water must be left over on the 31st December of the previous year to make up the deficiency between January to April, and at least 10,800 gallons on the 31st October to make up the deficiency of November to April of the succeeding year. Allowing for wastage and evaporation from the tank if the capacity is made 12,000 gallons, then the residual content of the tank on the first of each month will be as given at the foot of the statement.

Ground water supply—Hitherto, we have only dealt with the portion of rainfall which flows over the surface of a nearly impermeable area. But the crust of the earth varies from a state of almost complete impermeability to that of almost complete permeability. When rain falls on the earth gravitation tends to draw particles of water that enter the earth through the porous strata towards the centre of the earth and they percolate in that direction until they meet an impervious stratum, such as clay &c. where they are forced to change their direction of movement and follow along the surface of the impervious stratum towards an outlet in a subterranean

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* Statement is given in the previous page.

valley, and possibly to discharge into a lake, river or ocean. The amount of water that may enter the ground depends upon the following factors among others.

1. Amount of rainfall.
2. Capacity of surface to drain off the rainfall to the nearest valley or stream.
3. Porosity of the surface formation.
4. Permeability and topographical feature of the underlying strata.

Wherever there are pores or open fissures below the surface, water derived from rainfall is found at levels above the sea determined by the resistance of solids to its passage towards some neighbouring sea, lake or water course. Any such level is known as the *level of saturation*, and the plane of the surface of flow—the ***water table***. The water table is analogous to the flow line of a river surface, in that its profile depends upon the hydraulic gradient necessary to produce flow. The ground water table coincides with the surface water level of a stream at its margin. Passing back from the stream the water table rises, the slope varying according to the character of the underground formations offering resistance and also the quantity of water flowing through them.

The yield of a ground water supply depends on the amount of rainfall which will percolate into the ground in the catchment area tributory to the point of interception, on the method of interception of ground storage equalising the variation in the amount of rainfall which enters the ground, and lastly, on the capacity of the water-bearing strata to deliver its accumulated water to intercepting works.

Laboratory experiments through unassorted sands show that even a very slight scratching of the surface or the slight impinging of the one medium into another markedly affects the rate of flow. From the variety of conditions which prevail under the surface of the ground, especially in the alluvial formations of Bengal, it will be evident that no simple formula

can be devised to estimate the flow of ground water in a particular locality with any degree of accuracy. It is of course possible to obtain an approximate estimate of flow in one district under certain given conditions, and many such formulæ are available, but the results given by them err to the extent of 50 to 100 per cent.

Determination of yield—The best method of estimating the capacity of a ground water source is by actual pumping tests carried on for a sufficient length of time during dry weather to bring about a state of equilibrium between the supply and the delivery which will be shewn when the levels of water in the trial and indicator wells will remain unaltered. Pumping tests of short duration are apt to be very deceptive, as the ground water may exist in the form of a large basin or reservoir similar to a surface pond with small watershed and brief pumping test will be misleading like a similar test on a pond.

The yield that can be obtained from a percolation well or gallery depends upon the infiltration head and square foot area of sand tapped and also on the critical velocity, which latter again depends upon the size and specific gravity of sand. The *critical discharge* of a well or gallery is that amount of water drawn at a velocity which does not cause grains of sand in the water-bearing startum to be carried or blown into them. The infiltration head which produces this velocity is called *critical head* and the velocity which produces this conditions the *critical velocity.* If this head is exceeded in pumping, sand will flow into the well until the well is filled with sand to such a depth that no greater supply than the critical discharge can be obtained. If with the object of obtaining greater supply the sand is dredged out of the well and over-taxing continued, the well will again be filled with sand ; if this process is repeated for some time, a cavity will be formed round the well in the subsoil and either the ground round the well will sink or the well will collapse or sink bodily. It is for this reason that from a percolation well in a sandy soil more than

a certain definite quantity of water cannot be obtained, irrespective of the depth to which it may be sunk, and any attempt to get more discharge than this will endanger the stability of the well. Various means have been devised to overcome the blowing of sand ; sometimes an artificial layer of 3 or 4 feet consisting of broken stone and graded ballast has been placed at the bottom to weigh down the sand, but this produces only a temporary effect, for the sand is continuously drawn through the interstices of the plugging layer and soon chokes it up. It is for this reason that while running a test for a maximum yield, the well should be pumped to its critical head. The table 23 gives the critical velocity according to Theim :—

TABLE 23

Diameter of grain in inch.	Velocity of water in ft. per sec. to cause sand blowing.
0·0 to 0·01	0·0 to 0·10
0·01 ,, 0·02	0·12 ,, 0·22
0·02 ,, 0·04	0·25 ,, 0·33
0·04 ,, 0·08	0·37 ,, 0·56
0·08 ,, 0·16	0·60 ,, 2·60

The critical velocity should however be found out during the time of test by observing the head or discharge at which the sand starts blowing.

The trial well must be sunk to the full depth to which it is proposed to sink the permanent wells, and so also the indicator wells—especially those close to trial well from which the water is proposed to be pumped. Before pumping is commenced, if levels of water in different wells are taken, then the slope and direction of inclination of the water table can be easily ascertained. When pumping is commenced, the water level in the trial well from which the water is pumped will continue to fall until a level is reached, where the water-bearing strata under that filtration head can discharge into the well a quantity

equal to the quantity pumped. From this time the water level in the well will remain constant. An examination of the level of water surface in the different indicator wells will show that a dip has been formed in the water table, taking the shape of an inverted cone with its apex at the water level of the pumping well. The ideal surface of the cone will be in the form of a parabola which becomes steeper as it approaches the pumping well. The form of the cone depends upon the physical and geological character of the water-bearing strata and also the slope of the water table. The area inside which the water level dips down is called *circle of influence* which means that if more than one well is sunk within the area, the pumping of one will affect the yield of the other.

A pumping test is usually made by pumping at such a rate as to maintain a constant level in the well under test, provided the inflow is large enough to allow continuous pumping. If the pumping is continued for a long time, it will be observed that the circle of influence will gradually widen out and the yield will slowly decrease. This specially happens in a state of subsoil with fair porosity and flat gradient. To obtain reliable estimate of yield, the pumping test must be carried on continuously day and night for at least 7 or 8 days or until such a period when the water levels in the indicator wells remain constant in addition to the water level in the trial well itself. The maximum safe yield will be when the depth of lowered level in the well is substantially less than the infiltration head which produces critical velocity inducing blow of sand in the stratum. A quicker method of testing approximately the yield of a well is to sink a test well at the site where it is proposed to build permanent wells. The test well must be sunk to the full depth to which the permanent well is proposed to be sunk ; then it should be pumped with a view to lower the water level to such a depth as to just induce the critical velocity. When this level is reached, the pumping is to be stopped and the time taken to recoupe to its original level is to be noted. The safe yield per hour will be equal to the

quantity of water pumped out divided by the sum of the times taken to pump and recoupe.

Measurement of the Velocity of Ground Water flow— The rate of flow can also be estimated by the measurement of the direction and velocity of ground water. The direction of flow can be determined by sinking a number of indicator wells around the trial well. The time of flow between the trial and different indicator wells is first determined. The direction of flow is from the central well towards the well to which the time of flow is the shortest. The velocity of flow can be determined by introducing a colouring matter such as fluorescein or uramin in central well and observing the time taken to flow into the lower well. Fluorescein has a reddish orange color, but when dissolved in water it appears by reflected light as brilliant green. One part per 40 million can be detected by the naked eye and one part per 10 billion by comparison with a colorless standard.

Having determined the velocity, it remains to estimate the actual quantity flowing into the well or gallery. Of the total volume of body of sand or gravel in the water-bearing stratum, water will only fill up the pores amounting to about 25% to 33%, the porosity of the water-bearing stratum can be determined by collecting samples of materials by actual boring. The actual quantity that will be delivered will only be 25% to 33% of the water-bearing stratum tapped. If 'V' be the velocity of flow as measured, and 'A' the area through which the water is flowing, and if the porosity is assumed to be 33%, then the actual quantity 'Q' delivered will be:—

$$Q = 0{\cdot}33 \text{ V.A.}$$

For the purpose of rough estimate, sometimes engineers are required to find out the flow without any preliminary investigation, such as sinking of wells &c. The following table prepared by Turneaure gives the velocity of flow per day in different classes of soils. The information with regard to the slope of water-table and character of water-bearing stratum may be

collected from the records of wells or galleries in the neighbourhood.

TABLE 24.

Material.	Slope of Water-Table in feet per mile.					
	10	20	30	40	50	100
Fine Sand ...	0·2	0·4	0·6	0·8	1·0	2·0
Medium Sand ...	1·5	3·0	4·5	6·0	7·5	15·0
Coarse Sand ...	4·0	8·0	12·0	16·0	20·0	40·0
Fine Gravel free from Sand ...	20-40	40-80	60-120	80-160	100-200	200-400

In Midnapur, Birbhum, Bankura, and Shinghbhum districts of Bengal, wells and galleries constructed in beds of rivers yield about 30 to 40 gallons per hour per square foot of area of infiltration.

CHAPTER VI.

THE COLLECTION OF WATER.

General considerations—The works required for the collection of water for a public water supply vary very considerably according to the nature of the source, its topographical and geological conditions. The intakes and the works in connection therewith should be of such magnitude and located in such a position that it can give the quantity of supply required by a community and its future generation with a minimum probability of failure or trouble from the freaks or convulsions of nature, and that would require the minimum and least expensive treatment for purification to obtain the best results. These should be simple in design, easily accessible for operation and repairs, and sound and durable in construction. The different forms and arrangements for different classes of intakes are described in the following pages. The variety of intakes may broadly be classified into two, *viz.*, (1) Those required for surface supplies and (2) those for underground supplies. The first variety of intake may be subdivided again into (a) those required for upland waters—such as natural lakes or impounding reservoirs, (b) those for low land water—rivers or other perennial streams of considerable magnitude. The flow of a brook or river near its source is very spasmodic, and is not therefore to be depended upon as a source of supplies unless storage is provided for use in times of drought. Consequently, the arrangements of intakes in such supplies must be different from those required for supplies from perennial streams. With regard to the intakes for ground water supplies which are mainly from wells, springs and galleries, the form of structures and their arrangements are entirely different from each other, as the methods of collection in each of these cases are different.

Surface supplies.

Upland supplies--In such supplies an artificial lake or reservoir is formed by erecting a dam at a suitable site in the valley of the stream furnishing the supply either singly or in conjunction with another adjoining to it. The most important consideration is the choice of site which demands the greatest care and attention. In the first place, the site should be in such a position where the extent of drainage area yielding the storage is sufficient to furnish the requisite supply under abnormal conditions of rainfall, and it must be sufficently elevated to ensure gravitation of water to the area to be supplied and a proper pressure throughout the entire system of distributing pipes. The best site for an artificial lake is one at which the requisite quantity of water can be impounded on the smallest surface area with fairly steep sides, where the embankment retaining the water is the shortest and can have a solid and impervious foundation and can be constructed largely of materials available near at hand. Such a site will give the greatest capacity with the least cost, and the most uniform depth and thereby prevent growth of rank vegetation detrimental to cleanliness and purity of water. The geological conditions of the site should be such that there is little danger of leakage, either through the sides, or through the bottom of the dam. The underlying strata should be synclinal rather than anticlinal, for strata dipping away from the valley will drain the reservoirs of their contents. The site should be as far as possible free from cracks, fissures, or open faults, as the stopping of leakage through these have been found, in many cases, to be expensive and difficult. The most satisfactory site from a sanitary standpoint is where the watershed is uncultivated, uninhabited, and as far as possible free from herbage. The nearer these conditions are approached, either naturally, or artificially, the nearer an ideal site will be realised.

The form of retaining wall and the materials of which they are constructed depend upon the foundation on which they are built. Among the rocks, those of the igneous variety, granite,

porphyryte, basalt, trachyte &c., offer the safest foundation for masonry dams. Stratified rocks, however, do form a safe bed, less on account of their strength than on account of their position and cleavage. It is however, considered safe to build on them if the layers are compact and either horizontal or have a dip towards the reservoir. On the other hand, a clay stratum of at least 6 ft. in depth is generally considered sufficient to form a reservoir with an impervious bottom, especially in view of the fact that silt deposited later will increase considerably the water-tightness. Such sites are suitable for earthen embankment which need not be built on rocky grounds. Earthen embankments up to 150 ft. have been built and so far proved to be successful.

The Reservoirs have been formed of dams of various materials and of various forms of construction ; in this country only two types are usually used, viz., earthen and masonry.

Earthen dam—Earthen embankments are constructed with a puddle wall in the centre or on the water slope—some of the old embankments in Bankura and Birbhum districts have been constructed of homogenous material without any puddle wall. There is some difference of opinion as to the most suitable material for an embankment. The materials, of course, should be such that will ensure the water-tightness of the tank and offer resistance to slipping when under pressure. Regarding relative quality of clay and fine gravel as materials of embankment, Mr. W. J. McAlpin, President of the American Soc. of Civil Engrs. says :—"The particles of clay are cohesive and vein of water ever so small which finds a passage under or through clay is continually wearing a large opening. An embankment of clay is much tighter at first and is always liable to breakage."

"The particles of fine gravels have no cohesion. A vein of water first washes out from the gravel the fine particles of sand and the large particles fall into the space and these small stones first intercept the coarser sand and the next particles of loam which are drifted in by the current of water, and thus the whole mass puddles itself better than engineer could do with

his hand. The vacuities produced below by this operation are indicated by settlement on the top where more gravel etc., can be added as is found necessary. An embankment of gravel is comparatively safe and becomes tighter every day." Mr. Strange in his book on "Indian Storage Reservoirs with Earthen Dams" recommends the use of a mixture of soil composed of 1 part of pure black cotton or other clayey soil to 1 part of pure murum or shale.

Owing to the deficiency of the theory of stability of earthen dams, the cross section and the general form of their construction have been more or less standardized in almost every country. The following table giving the general sections for ordinarily good soils is taken from Mr. Strange's book mentioned above.

TABLE 25

Height of dam above Ground Level in Feet	Height of top of dam above High Water Level in feet.	Top width in feet.	Upstream slope.	Down-stream slope.	Width of dam at High Flood Level in feet.
15 and under	4—5	6	2 to 1	1½ to 1	20—23½
15 to 25 ...	5—6	6	2½ ,, 1	2 ,, 1	28½—33
25 ,, 50 ...	6	8	3 ,, 1	2 ,, 1	38
50 ,, 75 ...	6—7	10	3 ,, 1	2 ,, 1	40—45

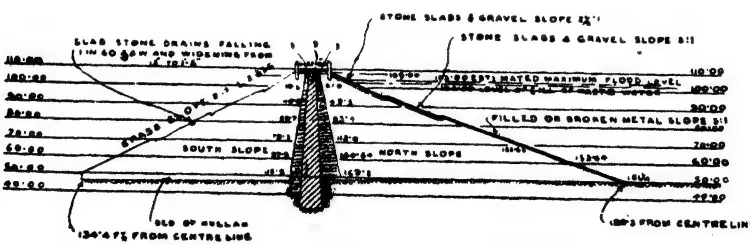
In American practice, a berm 5 or 6 feet wide is provided on the inner face—a few feet below the low water level and a wider berm on the outer face between one-third to one-half the distance from the top.

All angles in the cross section should be rounded off, as angles in earthwork can never be maintained. No matter, however well consolidated a dam may be, it will settle after filling ; therefore an allowance of 1/20 to 1/30 or more of its total height from foundation should be allowed in all works.

Core-walls—The necessity or otherwise of these walls in earthen dams has been a matter of controversy amongst water-works engineers, but certainly it is true that the general trend of opinion is in favour of their use. Theoretically, these walls are only necessary when an earthen dam owing to scarcity of suitable material for embankment cannot be made water-tight otherwise ; in practice, however, they have been found to be a good thing and when well constructed are worth their costs. The size and shape of these walls vary very considerably with the height of water retained and quality of materials. They are seldom made less than 6 feet wide at the top, at ground level one-third the height of water retained and 4 or 5 feet at the bottom of the trench which is taken to sufficient depth according to the nature of the subsoil to ensure water-tightness. The material used may be cement or lime concrete or selected clay free from roots, leaves or other vegetable matter and properly weathered.

There are several methods of consolidating an embankment such as hard rammers, road rollers, and carts loaded with soil ; for embankment, the last gives the greatest intensity of weight and seems to be the cheapest.

The Fig. 19 shows an earthen embankment for Bangalore water supply. The height of the dam is about 60 ft. above



Fif. 19.—Cross section of an Earthen Dam, Bangalore Water Supply.

ground and the section adopted has a top width of 21 ft. a road being carried over it, the front and rear slopes are 3 to 1 and 2 to 1 respectively. For want of suitable clay for puddle a wall

of lime concrete, composed of 1 part of mortar, ¾ part of gravel and 1¾ parts of broken stone, was built. The mortar was composed of equal parts of lime, soorkee and sand. The wall is 3 feet wide at the top and 14 feet at the bottom and is carried down to the firm rock in the centre and to the impervious soil near the flanks.

The inner face of the embankment is protected with a layer of stone slabs to prevent erosion by wave action.

Mansonry dams—The next class of dam to be considered is that in which the structure as a whole is so bound together that to a considerable extent it can be considered monolithic. This class of dam can only be constructed where natural hard bed-rock not liable to disintegrate can be reached at a reasonable depth below surface, and necessary stone for masonry can be quarried near at hand. The stone for hearting and face works of these dams should be sound, compact, heavy and as much impermeable as can be obtained. The actual construction of successful masonry dams varied very considerably (both in regard to the kind of masonry and mortar used) from cyclopean rubble to ashlar work. The dam is generally formed where suitable material is obtainable of cyclopean rubble ; stones up to 8 to 10 tons in weight have been used ; these are laid on their natural bed, the whole area of which is picked and dressed to a fair surface, so that it may be bedded properly, such stone should cover only a small portion of the width of the dam, and the spaces between them, where large enough, must be similarly built in with smaller but always as far as possible with the largest possible stone ; spaces too small for this kind of treatment are filled in and rammed with concrete. Each stone is thoroughly beaten down with heavy logs of wood to expel the air and until the mortar squeezes up all round. In some cases, the stones are dropped into a fairly thick bed of concrete and then worked down with bars. The faces of the work may be ashlar masonry thoroughly tied into the hearting. These works have been both built in cement and in hydraulic lime mortar ; in this country hydraulic lime mortar has been used in many cases. In the Jharia dam, designed by and

constructed under the direction of Mr. G. B. Williams, M.I.C.E., ghooting lime obtained from Gurpa near Gaya was used. The Tansa and Jubbulpur dams were constructed of hydraulic lime mortar. In construction, care must always be taken to deal with the fissure or springs. These should either be sealed if possible, or picked by pipes and carried to the outside of the wall as soon as can be conveniently arranged. But in case, the bottom of the up stream face of the dam is not watertight, a tongue wall 6 to 10 feet in width is built of suitable waterproof material along the toe of the dam.

STABILITY OF MASONRY DAMS—So much has been recently written on the subject of the stability of masonry dams that it is not possible to go into a detailed examination of the subject in a manual like this. A general outline of the principles and method of designing are proposed to be given.

A masonry dam may fail in any of the following ways:—

(1) It may overturn about its toe or the part above any horizontal plane may so revolve.

(2) It may slide as a whole on the foundation, or any part above it.

(3) It may fail by the crushing of the masonry or of the foundation.

(4) It may fail owing to the rupture of any joint due to tension.

Hence for a masonry dam to be perfectly stable, the following conditions must be complied with:—

(i) To prevent failure on account of (1) and (4)—The resultant of all forces acting at any horizontal joint must fall within the middle third of the thickness of the dam at that joint.

(ii) To prevent failure on account of (2).—It must by friction alone resist any tendency to slide on its base.

(iii) To prevent failure due to (3).—No masonry must be under more than the safe compressive resistance of the masonry or the foundation.

The conditions (1), (3) and (4) must be satisfied under all conditions of loading, *viz.*, when the reservoir is full or empty. Observance of these three conditions generally involves the fulfilment of the other, and therefore it is unnecessary to investigate—especially when the dam is built of cyclopean rubble masonry and all through joints are avoided.

Various methods and formulae have been proposed to find the cross section of the dam to satisfy the conditions of stability stated above ; any of them can be used in ascertaining an approximate profile, and then its stability can be tested by graphic resolution of forces ; if found defective, the profile is altered and graphic process is repeated until a cross section satisfying all the conditions of stability is obtained.

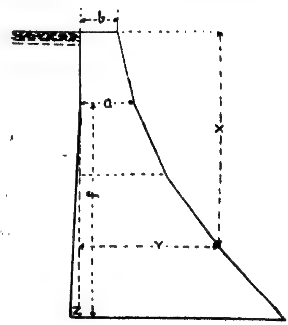


Fig. 20.

Courtney in his book on "Masonry Dams" gives the following formula* for working out the profile which is almost similar to one that can be obtained by Molesworth's formula.

H = Total height of dam in feet.

C = A constant of 7 to be used for all depths.

x = Depth below the surface of water in feet.

Y = Offset to the outter face of the dam from vertical line corresponding to the inner face at the top.

Z = Offset to the inner face from the same vertical line.

Then, $b = \sqrt{H + 2}$ ft.

$$a = \frac{H}{4} \times 0{\cdot}72$$

$$Y = \sqrt{\frac{0{\cdot}05x^3}{C + 0{\cdot}03x}}$$

$$Z = 0{\cdot}03H. \text{ or } \left(\frac{0{\cdot}09x}{C}\right)^3 \text{ for all depths.}$$

* The notations in the formulas refer to fig. 20.

Having thus determined the approximate profile and drawing it out on a sheet of paper, the outline may be altered to give a cross section which combines stability and elegance in form. The outer face of the wall is generally made into a curve and at the same time passing through the obligatory points obtained by the formula. The inner face is generally given a straight batter. After this is completed, the next step is to ascertain the line of resistance or pressure with reservoir full and empty. In order to proceed, it is necessary to settle about weight per cubic foot of masonry to be adopted. The weights of masonry in dams actually constructed vary from 125 to 164 lbs. per cubic feet. The stones in construction should not have a specific gravity of less than 2·5. The weight of a cubic feet of masonry can be found by finding out the actual weight of stone proposed to be used on the work and assuming that the masonry will consist of two-third stone or one-third mortar. The following weight of masonry is given in Baker's "Masonry Construction".

TABLE 26

Brickwork (pressed brick thin joint)	146 lbs. per cft.
,, (ordinary quality)	125 ,, ,, ,,
,, (soft brick thick joint)	100 ,, ,, ,,
Cement concrete (best)	160 ,, ,, ,,
,, ,, (porous)	130 ,, ,, ,,
Granite or lime stone (well dressed)	165 ,, ,, ,,
,, ,, ,, (rubble well-dressed with mortar)	155 ,, ,, ,,
,, ,, ,, (rubble roughly dressed with mortar)	150 ,, ,, ,,
,, ,, ,, (rubble well-dressed dry) ...	140 ,, ,, ,,
,, ,, ,, (rubble roughly dressed dry) ...	125 ,, ,, ,,
Mortar dried	100 ,, ,, ,,

The next point of importance, that is to be considered, is the limit of pressure on the masonry to be adopted. The maximum pressure in the masonry of dams built, appears to vary from 4 to 12 tons and the average being about 6 tons per

sq. foot. The limits of safe pressure assumed by Baker are as given in the table below:—

TABLE 27

Concrete ...	...	5 to 15 tons per sft.
Rubble ...	...	10 ,, 15 ,, ,, ,,
Squared stone	...	15 ,, 20 ,, ,, ,,
Lime stone ashlar	...	20 ,, 25 ,, ,, ,,
Granite ashlar	...	30 ,, ,, ,,

Having settled the weight of masonry and the limit of pressure to be allowed, the next step is to divide the profile into convenient number of sections, to calculate the area, weight and centre of gravity of each section, and find out the direction and intensity of resultant on the bottom of each section. The way in which this is done is clearly shown in Fig. 21 for masonry dam. The centre of gravity of each of the sections is found in the manner shewn in Fig. 22, and that of two or more sections combined by means of *polar diagram* and *funnicular polygon* as shewn in Fig. 23. To obtain the points in the line of pressure when the reservoir is empty, it is only necessary to drop perpendiculars from the centre of gravity of sections, sections 1 and 2 combined, sections 1, 2 and 3 combined and so on, on the joints of the base of different sections. The magnitude of water pressure acting on the face of the dam above different joints are then calculated and set off at one-third the depth for the section concerned. Then, these pressures are combined with the weights of the corresponding sections. To obtain points in the line of pressure for reservoir full, it is only necessary to draw from the points of intersections of the resultants with the corresponding joints of the dam. As stated before, if the condition of no tension at any joint is to be observed, then both of these lines of pressure must be within the middle third everywhere.

For the purpose of finding the distribution of the pressure over a joint, the average intensity of pressure, *i.e.* $\frac{\text{total pressure}}{\text{area of surface in sft.}}$ is set off to scale at the centre of the joint which is divided into 3 equal parts. Then it is proceeded in

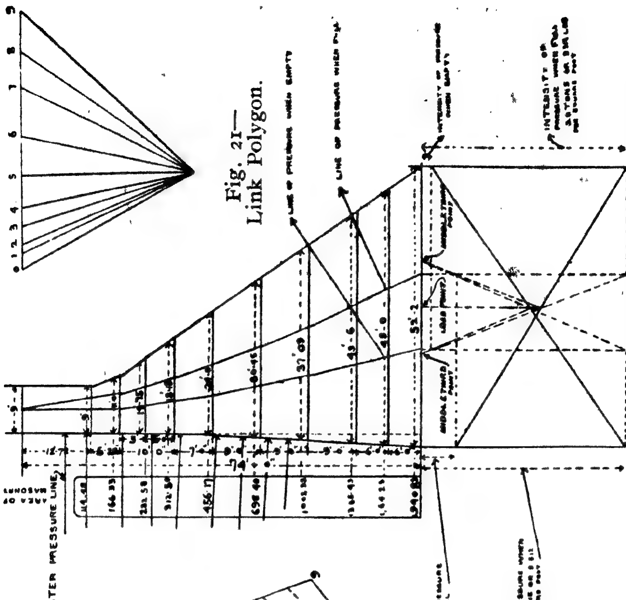


Fig. 21—Link Polygon.

Fig. 21—Graphic determination of stability stresses in a masonry dam.

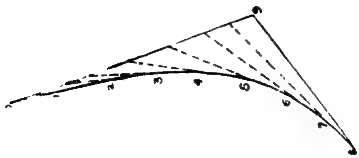


Fig. 23.—Funnicular Polygon.

Fig. 22—Grapical Determination of C. G. of any Section of a Dam.

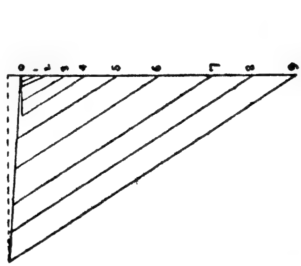


Fig. 23—Link Polygon for combining Water Pressure and Weight of Masonry.

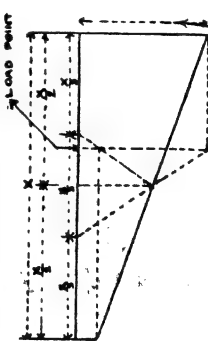


Fig. 24—Graphic Determination of Intensity of Pressure on Foundation.

the manner shewn in Fig. 24. The intensity of pressure at either end can as well be calculated from the following formula :—

$$I=\frac{W}{x}\left(1\pm\frac{6d}{x}\right)$$

where

I = Intensity of pressure.

W = Total Pressure.

x = base width of the wall.

d = The distance of the point of intersection of the base and the line of thrust, *i.e.*, load point from the centre of the base.

It may be observed here that the manner of distributing pressure considerably affects the crushing strength of the material. An uniformly loaded material may have 50% more crushing strength than one with eccentric loading.

The above is the ordinary method of investigating the stability of a masonry dam. Recently, however, attention has been directed to the stresses on planes other than horizontal. M. Lavey showed in a paper on the masonry dam that the vertical pressure at the upstream end of any joint calculated by the law of uniformly varying stresses should not be less than that of water pressure at that joint with a view to prevent intrusive water getting into the structure. Subsequently, in the beginning of this century, Messrs. Acherly and Pearson in a paper *"On some disregarded points in the stability of masonry dams"* attempted to extend to the vertical section of a dam the method generally accepted as applicable to horizontal sections. It is further contended that stresses in vertical section are more important in the investigation of stability of dams. These theories, however, are not generally accepted by the profession, as many large dams have been constructed during the end of the last century in various parts of the world, all designed in conformity to the theory of middle third, and no notable instances of failure have been heard.

In some cases, the dams are made curved on plan. Prof. Rankine in a report on masonry dam remarks as follows:—"As regards the effect of giving the wall a curvature on the plan convex towards the reservoir, I look upon this as a desirable, and in many cases essential precaution in order to prevent the wall from being bent by the pressure of water into a curved shape concave towards water, and thus having its outer face brought into a state of tension horizontally, which would probably cause the formation of vertical fissures and perhaps lead to the destruction of the dam. I consider, moreover, that the calculation of stability, which treats the dam as an horizontal arch, is so uncertain as to be of doubtful utility; and I would not rely upon them in designing the profile".

Reservoir Accessories—In every impounding reservoir, whether formed by masonry dam or earthen embankment, the following accessories are always necessary.

(1) CATCHPIT—Some sort of arrangement for arresting silt and other substances getting into the tank and thereby preventing the silting up of the tank. This may be done by building a sort of settling tank at upper end of the reservoir and allowing only comparatively clear water to overflow into the tank. At the Jharia reservoir, a road round the tank has been constructed with a *catchpit* at every water course feeding the reservoir.

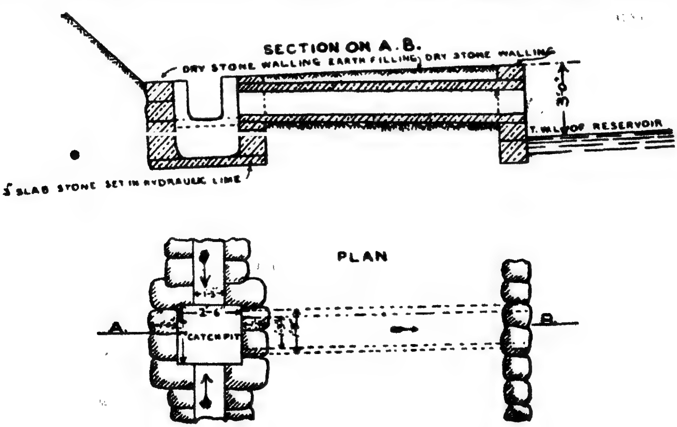


Fig. 25.—Details of Catch pit.

The catchpits are placed on the hillside of the road, and only water comparatively free from silt are allowed to overflow into the tank. Fig. 25 shows the details of the arrangement.

(2) WASTE WEIR—The safety of a reservoir depends in a great measure on the proper proportioning and construction of waste weir by which the surplus water that cannot be stored in the tank can be speedily discharged. The first step in designing such work as explained before is to ascertain the maximum probable flood discharge of the watershed and make the length of weir such that it can pass off this entire volume without allowing the high water level of the reservoir to rise above the level proposed in designing the dam.

The best site for such work is—where from the configuration of the ground a low gap occurs in the line of hills surrounding the tank, which by reasonable cutting can be brought to the level required for the weir. When such site is not available, the waste weirs in case of earthen dams are built at one of the extremities of the embankment, where it should be formed by a cutting into the solid ground and not by passing it over any portion of the embankment, from which it must be always separate.

In the case of masonry dams, however, the dam may be constructed as a weir dam, so that the flood can flow over the top of the wall. This form of construction is shewn in Fig. 26 and was adopted in Jharria water works scheme.

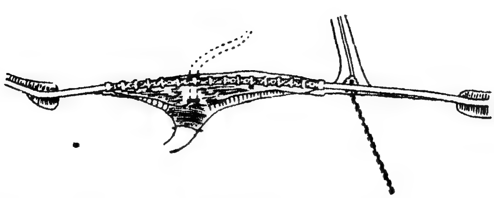


Fig. 26.—Plan and elevation of Jharria Dam.

(3) SPILLWAY—The function of this channel is to take away the flood water falling over the waste weir in such a manner as will be least injurious to the stability of the dam. On the grounds of economy and safety, it is desirable to discharge the flood as quickly as possible, but the velocity in the channel must not be such as to lead to the destruction of dam by scouring away its foundation. The high velocity of the channel can be avoided by constructing it in the form of a series of steps and making the tread of each step in such a way as to form a water cushion.

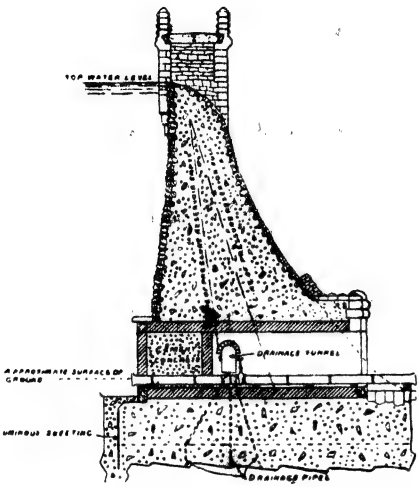


Fig. 26.—Cross Section of Jharria Dam through outlet culvert.

(4) OUTLETS—Where an impounding reservoir has been formed by earthen embankment, it is desirable for the security of the dam to build an intake at one side of the tank, and as far as possible, away from the end of the embankment with discharge culvert built in a tunnel on one side of the valley taken sufficiently below the toe of the embankment. In case of reservoirs formed by masonry dams, the same course may be adopted as an additional precaution, or it can be built alongside the dam. The arrangement of the outlet should be such that the comparatively clear water near the surface can be drawn under varying depths of water in the reservoir, and screens should be provided to arrest floating matters and aquatic animals flowing into the supply pipes. It is also necessary to make arrangements for the periodical cleansing of the screen, which often gets choaked up with fine deposits from outside. The founda-

tion of such work must be carried down to solid ground so that no settlement endangering the stability of the dam can take place. Fig. 27 shows the valve tower outlet of the Jharria dam.

In reservoirs of small depth —say up to 25 ft., a valve tower may be dispensed with, and a syphon of air-tight pipe may be constructed. It may be taken over one end of the dam to deliver to a point sufficiently away from the outer toe of the dam. The inner end of the pipe should be higher than the outer one, both ends must be protected by valves, and a cage of screen should be fixed to the inner end. Valves may be put in at any intermediate position for drawing off water from the surface. These may be worked by means of spindles taken above the water level of the reservoir. When a masonry dam is built in a narrow valley, a washout pipe with valve may be fixed inside the outlet culvert to enable the silt to be removed by scouring.

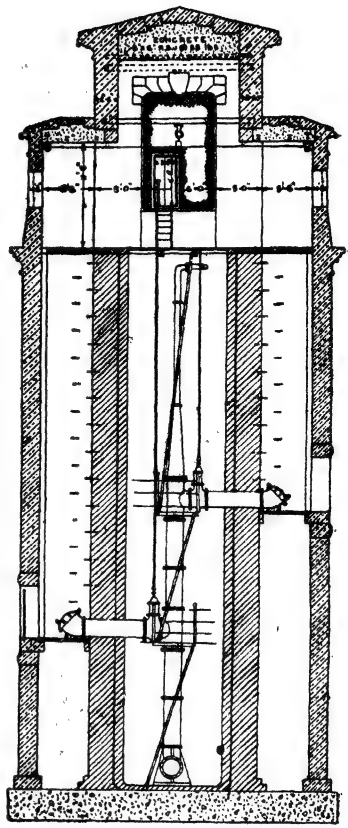


Fig. 27.—Valve Tower outlet.

In connection with this, it has been discovered that it is frequently impossible under certain heads to open such scour valves to their full extent on account of the vibration that sets up in the valve, and also the danger which may result to the toe of the embankment due to the scouring action of the issuing jet. This has now been overcome by what is known as a *Jet Disperser*, and the illustration on Fig. 28 shows a 42 inches scour valve at the bottom of an embankment with a disperser attached.

The water is issuing under a head of 75 ft., and although the ground in front of the toe is of soft and loose material, the issuing water has no result on the bed of the river. Previously, it was impossible to open any of these scour valves more than a few turns.

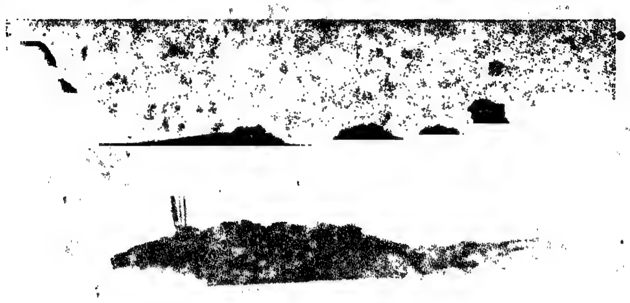


Fig. 28.—Jet disperser.

The design of the disperser is based on the hydraulic properties of the free vortex, and is such that it shatters any size of jet, travelling at any velocity, into a conical shower of drops.

Low land or river supplies—The supply from rivers in this country gives little cause of anxiety owing to the storage of water, as they almost all have a sufficient flow throughout the year to supply a town without storage. The real difficulty is the liability of pollution, for which not only filtration, but also constant watchfulness and care, is necessary. The intake should be in such a position where the probability of direct pollution is remote, and where the course of river is more or less permanent, as Indian rivers generally follow erratic course—especially in the alluvial plains of Bengal. The intake should be at a narrow part of the river where the flow is more rapid, the channel more deep and consequently, a better quality of water can be obtained than from a part where the water is comparatively stagnant. The level of the intake or suction pipe should be 3 to 4 ft. below the lowest water level of the river, and a screen should be provided in front of the pipe to prevent

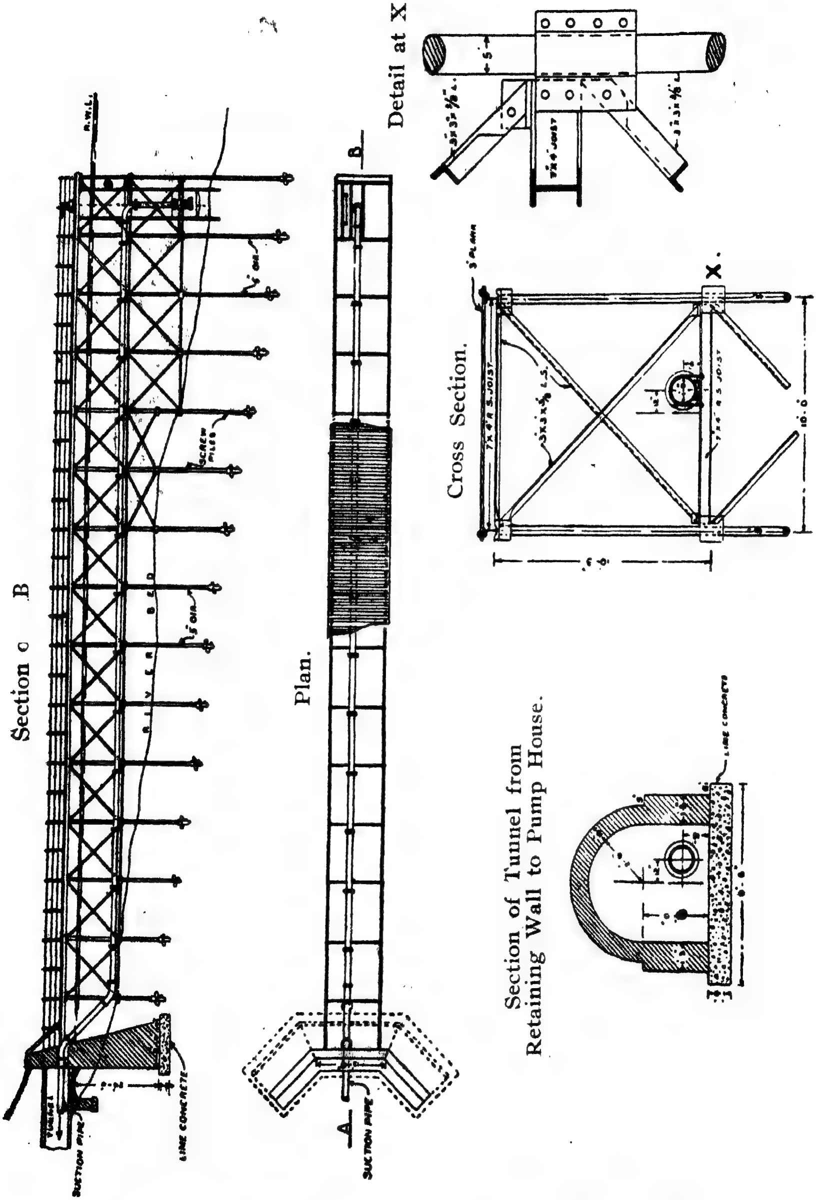


Fig. 29.—Intake of Hooghly-Chinsurah Water Works.

floating matters entering the pipe. The intake or suction pipe is generally carried on a jetty built into the river, and to make the jetty as short as possible, the intake should be at a point where the main deep channel is as close as possible to the bank. It should also be investigated if the bank is liable to erosion, as this may require heavy expenditure on protective works. Fig. 29 shows the intake for a river supply at Hooghly-Chinsurah. In case, where the supply has to be pumped from the river, a rose piece and a foot valve at the intake end of the pipe is often necessary. This should at least be 3 to 4 feet above the bed. The intake of a supply from an excavated tank is similar to that from a river. Fig. 30 shows one for Sibpur College water works.

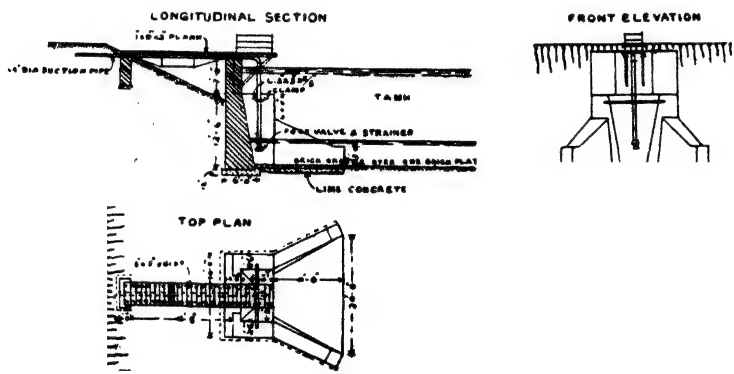


Fig. 30.—Intake for a tank supply.

Ground Water Supply.

General Consideration—The growing importance of ground water supplies in this province induces us to deal with the subject in some detail.

The method of collection of ground water depends considerably on the geological condition, position with reference to the surface, and capacity of the stratum to be tapped. Rain water percolating through a pervious stratum descends until its downward motion is arrested by another underlying impervious stratum, and then it follows the inclination or dip of the other stratum to the lowest point of its outcrop where it

emerges as a spring. Only a deep-seated spring or one in which the porous stratum is overlaid with an impermeable stratum is suitable for domestic supply. The purity of water from such a source is due to the thorough filtration the water has undergone in passing through the porous stratum before appearing on the surface as spring. The yield of a spring depends upon the area and rainfall of the watershed drained by it, the flow from it being greatest in wet season.

There may be such geological condition, however, where a water-bearing stratum is available within a short distance from the ground surface although no spring is formed. In such cases if this porous stratum is composed of coarse materials, a well sunk into it will admirably answer the purpose.

In other instances, where this pervious stratum is at a considerable depth from the ground surface, and sinking of a masonry well becomes either expensive or impracticable, it becomes necessary to sink tubular wells to the stratum. Filtration galleries are built in some instances, where it becomes necessary to intercept ground water from a greater area than can bè influenced by a few wells. They are in fact elongated wells run horizontally and usually at a small depth. There are instances in which the main supply from wells are augmented by galleries or by driven wells extending from them into the porous strata at lower depths. It is evident, therefore, that one plan of collection cannot be applicable in all these cases.

Springs—In the development of a spring supply, the following principles should be observed in the design of the intake chamber :—

(1) The chamber should be designed in such a manner that it will be able to intercept all the water the spring can yield. At the same time, care must be taken to prevent pollution from surface water.

(2) That the water is clear before it enters the outlet pipe ; to accomplish which, it may be sometimes necessary to provide settling basins or an artificial filtering medium.

(3) In cases, where the permeable stratum of the spring is

not overlaid by an impermeable stratum, the porous grounds at or about the spring should be well protected from contamination.

(4) The water from the spring can be collected through some small openings of suitable size in the rear wall backed by graded stone and gravel to keep back the sand.

(5) Every outlet pipe from a spring should be provided with a strainer at its mouth and be 1 to 3 ft .above the bottom of the chamber to prevent solid matters entering the pipe.

(6) The chamber should be provided with an overflow and washout pipe to drain out the surplus water and sand collected in the chamber. A ventilating shaft and a manhole are also necessary.

(7) It is desirable to have some means of determining the flow from the spring, and for this purpose, the chamber is made large enough to permit the construction of a temporary or permanent weir.

With sufficient precaution against surface pollution, springs often provide a valuable source of water supply—being usually free from organic impurities and equable in temperature. When their flow is limited, a collecting gallery may be run into the porous stratum to tap larger area of stratum. The common form of intake with necessary accessories is shewn in Fig. 31.

Wells—Wells may be classed into three classes, *viz.*, shallow, deep and artesian, according to the geological feature of the ground in which they are sunk. When a well draws its supply from a porous stratum adjacent to the surface without any intermediate impermeable stratum, it is called a *shallow well.* In this case, the plane of surface saturation is at no great depth, and the well is sunk only a few feet below the lowest level of this plane. As the rain water, owing to its highly solvent nature, carries with it any impurities, more particularly of an organic nature, met on its way, and as the distance, through which it has to pass through before it is drawn out, is very short, supplies from such wells are liable to be dangerous for use for domestic purposes. Only in cases, where the well can be sunk in places far away from any possible source of pollution, can the use of water from such wells be safely allowed.

Deep Wells—These are wells sunk through an impermeable layer to a water-bearing lower stratum lying over another impermeable layer beneath it again. The terms *deep* and *shallow* wells do not actually refer to the depth to which they are sunk. A shallow well in practice may be actually deeper than a *deep* well sunk at a different place. The danger of pollution to which shallow wells are subjected becomes considerably less in this case owing to the impermeability of the soil above it, and to the infiltration through a much longer depth of sand. But, at the same time, water from such a well, owing to the prolonged contact with strata, absorbs a considerable amount of mineral matter during its passage. It is for this reason that they are usually harder than those obtained from *shallow* wells.

Artesian Wells—These are wells sunk in such a locality where the line of slope in the water-bearing stratum is above the surface of the ground. The water in these wells rises up above the surface and overflows the well owing to hydrostatic pressure transmitted through the porous stratum. Such a well must necessarily be located in a valley or sufficiently low ground in relation to the surrounding watershed area, from which the supply is drawn, so that the line of saturation in the permeable stratum to which the well is sunk may be well above it. The artesian condition of well therefore depends upon the complete enclosure of porous stratum between two impervious strata,--the height of outcrop of the porous stratum through which the rain water passes in relation to the site where the well is sunk and the continuity of the water-bearing stratum from the well to the outcrop. So far artesian condition has not been discovered at any place in this country ; it must not necessarily be supposed that such geological formation cannot be found. Water from artesian wells is generally of the same quality as from deep wells. A deep well is generally a sub-artesian well, as the water from this well in many cases rises above the level of the impervious stratum over the water-bearing stratum tapped, though not over the ground surface.

Wells used for water works purposes are generally of two forms of construction, *viz.* (1) wells of large diameter and compara-

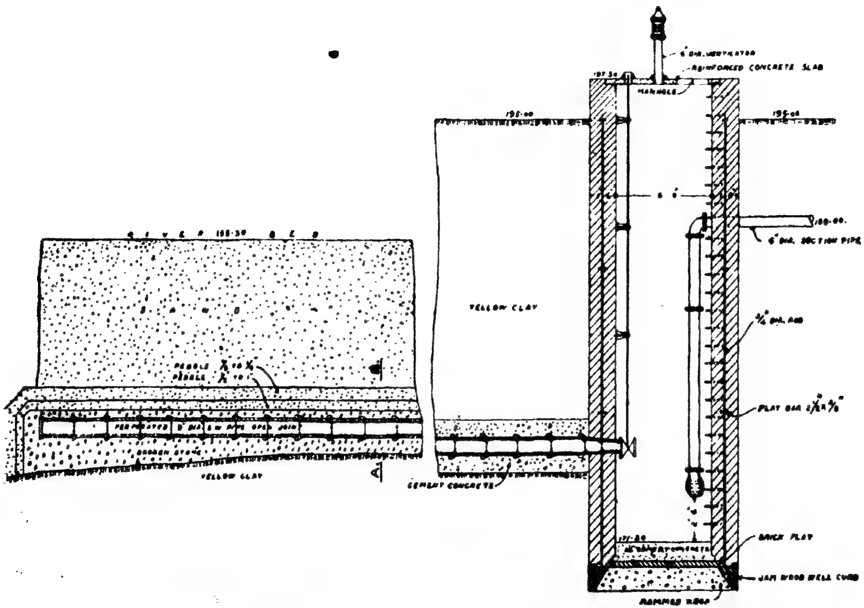


Fig. 32—Intake for a filtration gallery for supply—
Suri Water Works.

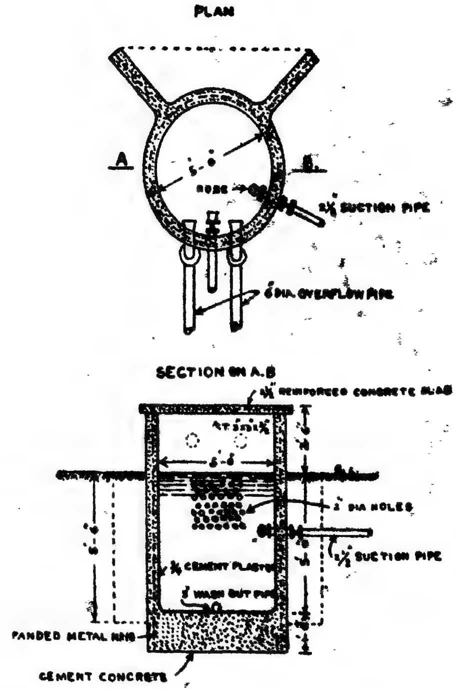


Fig. 31. Jharriah Intake.

tively of shallow depths and (2) of small diameter driven to considerable depth. Large shallow wells sunk in sand in the bed of rivers or near their edge are liable to furnish a supply of genuine ground water, but in some season, more or less mixed with less desirable substance from the flowing stream above or near it. The sites of these wells should be such that the chances of direct pollution are remote, and they should be well above a town or bustee, and a valley or khal draining into it.

Masonry Wells and their Accessories—The construction of small wells is a comparatively simple matter—especially when they are lined with brickwork. An open excavation is generally first carried down to the water level, where the curb is laid level and the steining is built up to a height of about 2 to 3 ft. above ground surface. The curb with its load of brickwork sinks as the excavation inside the well proceeds. The steining is then built up to its former level and the excavation is carried down and the process of sinking is repeated. Care should be taken to sink the well as uniformly as possible, as otherwise the steining is likely to crack or collapse. For this reason, care should be taken in constructing steining to complete each course of brick all round in order that the curb may be equally loaded on all sides. The friction of the earth against the masonry sometimes prevents the sinking of the well ; this difficulty is overcome by adding extra load on the top of the steining and also by cutting away the earth below the steining from inside. The curbs for wells of small diameter are generally made either of *babul, jam* or *simal* wood, the last two have the reputation of practically being indestructible under water. For large diameter wells, reinforced concrete and steel curbs are now generally used in Bengal. Bond bars and bond flats are used to tie down the different courses of brickworks, so that they may not crack during sinking. The steining is made of brickwork in hydraulic lime and made as watertight as possible to prevent pollution by infiltration, and for this, the inside of the well is plastered with cement. Iron linings have been used in other countries for this purpose. The steinings for wells up to 10 ft. diameter are made 15 inches thick, for wells from 10 to 15 ft. diameter

20″ thick and for diameters from 15 to 20 ft., a thickness of 25″ should be sufficient. For wells of 30 ft. diameter, 2′ 6″ steining has generally been used. These thicknesses are for wells up to 30 ft. depth, and if the wells are sunk deeper than this, the thicknesses should be increased by another 5″ for every 10 ft. depth.

The cost of sinking and constructing wells per foot of depth in this presidency varies from about Rs. 7/- to 10/- per foot of diameter.

The following accessories are necessary for a well for public water supply:—(1) A covering with a ventilating shaft carried above flood level, when the well is in river bed, and (2) when the supply has to be pumped, a suction pipe with foot valve and strainer is also necessary. The general arrangement for a supply from a well is shewn in Fig. 32.

Small Diameter Wells—These are of three types, *viz.* (1) Driven wells with closed end, (2) driven wells with open ends, and (3) bored wells. Of these, the first type was used in the Abyssinian war for temporary water supply for the army and are known as Abyssinian well. These are generally adopted for use in soft ground and up to a depth of about 60 feet. A well of this type consists of iron tubes from 1¼″ diameter to 6″ diameter in sections of 3 to 10 ft. which are driven into the ground. The bottom section, which is perforated and fitted with a brass strainer, has got a steel point to enable it to penetrate into the ground. As the tube is forced into the ground, a new section is screwed on to the upper end of the last tube until the required depth is reached. The tube may be driven by an wooden monkey and guide or other convenient means. For the purpose of driving, a cap is usually fixed on the top end of the tube so that the tube may not be damaged by the blow of the monkey. Chockage is removed by forcing a jet of water through a half-inch tube until the well is fully cleared. Then, a pump is fixed over the composite pipe and the work is completed. This type of well is useful for a limited supply for an isolated house and in favourable conditions of soil; they are hardly of any value for a municipal supply.

Driven Wells with Open Ends—These are suitable for comparatively harder ground and also for a larger supply. In Bengal, they are sunk by what is known as *hydraulic wash method*. The sinking is done by removing the material by means of a water jet, and at the same time driving the well by means of a monkey or pile driver. These wells consist of steel tubes in lengths of 6 to 10 ft. each as in the other case. The first length having a steel shoe attached to its lower end is first forced into the ground and a pipe of much smaller diameter is inserted into it. To the lower end of the latter a steel perforated point is welded, so that when a jet of water is forced through, the small pipe can discharge through the holes and play on the point of the steel shoe. This loosens the material about the point of the tube and a large part of which is forced out by overflow from the pipe. At the same time, the driving is done by means of a monkey or pile driver until nearly the whole of the first length is sunk. Then, the successive lengths are screwed and the process is repeated until the level from which it is desired to obtain water is reached. Extra strong pipes called 'boring tubes' are generally used for such wells, care being taken that the joints are screwed up so that the ends of the pipes are in contact and the successive lengths are in one line and truly vertical. To loosen the grip of the soil, the well tube when being driven is turned round by means of pipe-tillers.

The lower portion of these wells may be perforated with small holes, when the water-bearing stratum is composed of coarse material. When, however, sand is met with, the holes are usually covered with suitable gauze. Different companies have got different methods of sealing the bottom. The best and surest method of doing it probably is to sink the well deeper by the length of the strainer. Then, a proper strainer with a lead bevel packer fixed on the outside of the top end of the strainer, can be dropped into position, and the tube is withdrawn nearly to the top of the strainer.

Bored Wells—In this system, a vertical hole of a diameter larger than the diameter of the well is first worked in the ground to the full depth to which the well is proposed to be

sunk, and then the well tube with strainer is dropped into position, and the outer casing is removed. This system can be adopted to almost any condition of subsoil and to any depth. The method of boring adopted in this country is either *hydraulic wash* or *percussive or Chinese* method. In the latter method, a tool similar to flat or V chisel connected to iron rods having screwed joints at ends are made to fall freely and frequently on the bottom of the hole and a circular motion is given at the surface twisting the tool through a portion of the circle at each stroke.

The vertical reciprocating motion is given to the iron rods to break up the material at the bottom of the borehole and penetrate the strata. The circular motion at the surface is given in order to ensure, that the tool does not strike the same point at each stroke and also to make the hole circular.

When sufficient material has been cut away, the chisel is replaced by a tool called "auger nose shell" and worked up and down in a similar way to the boring rod, and the debris cleared out. A valve at the bottom of the sludger opens and closes with reciprocating motion of the rod and thus enables the pulverised materials to work inside the barrel of the sludger but not to escape when once got in.

The tools used in this system are of endless variety. Some examples are shewn with their description in Fig. 33.

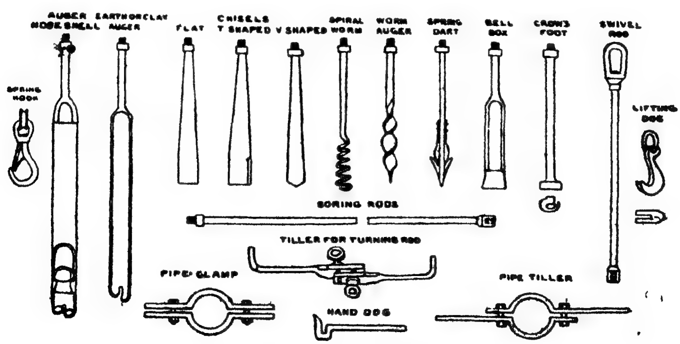


Fig. 33—Boring Tools.

Certain preparations are necessary as a preliminary to the commencement of boring. A well about 5 or 6 feet in diameter is usually first dug to a depth of 6 or 7 feet. Above this, shear legs are erected so that the rope passing over the pully at the top may hang over the centre of the well. A temporary platform is built on the top of the well so dug for the workmen employed in screwing and unscrewing the rods and pipes and also for giving them a circular motion. The workmen employed in twisting the casing tube are posted inside the dug well, and those who work the windlass for the reciprocating motion of the rod are stationed on the surrounding ground.

The time and trouble spent in lifting the boring rods from great depths each time the chisel is replaced by a sludger or *vice versa* forms a serious drawback in this system.

When geological conditions permit, it is therefore generally replaced by hydraulic washing system.

The casing is generally a solid drawn steel pipe with screw joints, and is much thicker than an ordinary pipe. The coupling is made by a sleeve joint, by a flush joint, or by an inserted joint. In first two cases, one end of each pipe is expanded and threaded inside thus practically forming a sleeve. These joints are shewn in Fig. 34.

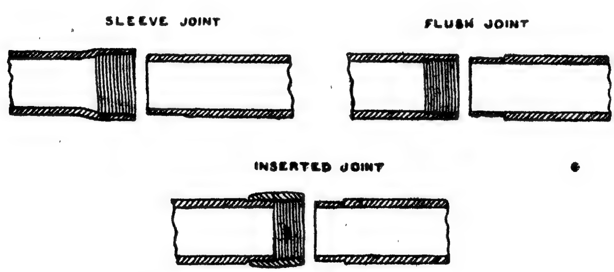


Fig. 34—Joints for boring pipe.

Strainers:—The discharge from a tube well depends upon the nature of the water-bearing soil and on the length and perforation of the strainer. The resistances to flow into a tube well are as follows:—

(1) Resistance to entry.

(2) Frictional resistance of the tube.

Resistance to entry depends upon the infiltration head, and the total area of perforation. The total area of perforation should be sufficiently large to keep the velocity of flow down to 2·5′ to 5′ per minute to prevent sand from entering. Mr. Brownlie recommends ½ inch per second in the Punjab sand and gives the following delivering capacity of his patent strainer for different sizes of tube wells.

TABLE 28

Internal diameter	3½	5	5	7	7	9
Lengths of strainer	34	42	54	54	74	95
Discharge in cusecs	·25	·5	·75	1·0	1·25	2

He fixes the infiltration head at 7 ft. for 5″ well and 14 ft. for 10″ well as most suitable for the Punjab sand.

Another kind of strainer largely used in this country is the Ashford strainer. Mr. Ashford fixes the length of the strainer at 150 times the diameter of well and infiltration head up to 20 ft. which is said to have no deleterious effect on the flow. The following table gives the discharge of wells sunk in the Punjab sand.

TABLE 29

Diameter of well ...	5	5	5	7	7	7	10	10	10
Length of strainer ...	35	50	100	50	75	100	50	100	120
Discharge in cusecs ...	·33	·5	·9	·65	1·00	1·12	1	1·8	2

A third type of strainer coming into use in this country is Johnson strainer, which is considered strong and very suitable in water-bearing stratum of alluvial sand. The following particulars with regard to this type of strainer may be useful in deciding the lengths of strainer.

TABLE 30

Diameter of Well ...	3	5	6	7	8	9	10
Clear opening through Screen in inches.	2·000	3·875	4·750	5·625	6·625	7·625	8·625
Lineal ft. of inlet per foot of screen.	48	82	100	113	130	147	164

This strainer is shewn in Fig. 35.

In Bengal, tube wells are generally provided with sufficient length of strainer to deliver the requisite supply at the approximate rate of 1 to 1·5 gallons per minute per sq. foot of strainer with 10 to 14 ft. infiltration head. Chart II gives the delivery of Johnson strainer under different working conditions.

The following extracts from the specifications of tube wells prepared by Mr. W. Kiersted are here given as a suggestive for that of strainer.

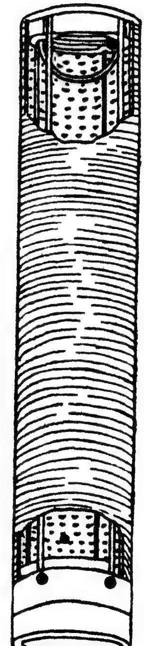


Fig. 35.
Johnson Strainer.

"Each well shall have a strainer of perforated brass or of other equally as good non-corrosive material acceptable to the Engineer, of such length as shall be best suited to depth and character of the sand stratum. The clear strainer opening must have a combined area of not less than 10 times the inside area of the strainer. The size of the openings shall be graded with respect to the coarseness of the sand in such a manner as to exclude sand but freely permit the flow of water. The thickness of the metal at any point in the body of the strainer must not be less than ⅛" and otherwise the strainer must be sufficiently stiff for the work. There shall be a metal plug at the

bottom completely excluding all sand. The strainer shall be attached to the casing by watertight joint."

Spacing of Wells :—When more than one well is required for a supply, the arrangement of spacing of wells should be such that working of one does not cause diminution in the discharge of another adjoining it, and also the maximum amount of ground water can be drawn from each. The maximum discharge can be obtained by placing the wells in line at right angles to the direction of maximum slope of ground water table so that almost equal heads in different wells can be secured. In the fine sand stratum of Bengal, the mutual interfering capacity of wells is limited as only a restricted amount of water can be drawn. In any case, they should not be spaced closer than the depths to which they are sunk.

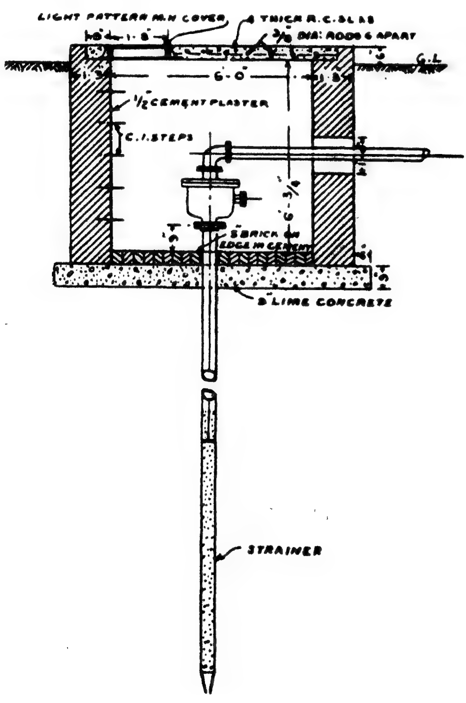


Fig. 36—Intake supply for a Tube Well Supply—Faridpur.

Fig. 36 shows the arrangement of intake of tube well at Faridpur. A considerable difficulty is often experienced in a well plant from the accumulation of air. Some air enters the well with the water, but most of it probably leaks through the joints of the well-casing or suction pipe. A line perfectly water-tight under hydraulic pressure may nevertheless leak when tested under air pressure. For this reason, great care should be taken to make the wells and pipes as air-tight as possible.

There is less trouble in plants with small suction pipes running down inside the well than in those where the well tube forms the suction pipe of the pump.

It is most important that wells of this nature should not be over-pumped, and once their safe yield is determined, means should be taken to prevent any possibility of over-pumping. This is managed by putting in a Disc Type Venturi Meter on the delivery pipe. The diagram paper has the maximum discharge marked in red and the pump attendant must see to it that this rate of pumping is never exceeded.

This system of preventing over-pumping is now being largely adopted, and one of the most recent schemes, in which this Venturi Meter was put to the use, was in connection with the tube wells for the Puri Water Supply. An illustration of this type of instrument is shewn in Fig. 37, but in this case no index is required to shew the total quantity, although this can be added if considered desirable.

The cost of tubewell sunk in Bengal by hydraulic wash is given in the table below:—

TABLE 31

Description of Tube Well.			Cost.	Cost For additional 25 ft. beyond Specified depth.
Diameter of well in inches.	Depth in feet.	Length of strainer used in feet.		
			Rs.	Rs.
1½	250	25	1,500	150
2	250	30	1,800	175
3	300	60	4,000	250
4	300	60	4,500	300
5	300	60	6,000	375
6	300	60	7,000	450
7	300	60	9,000	500
8	300	60	12,000	600
9	300	60	16,000	700

SHROUDING WELLS :—A method of construction known as *'shrouding'* has recently been introduced to prevent drawing of fine particles into wells by surrounding the strainer with a layer of gravel. The success or otherwise of the method is still not very certain. A method of construction adopted in America is given by Babbit and Donald as follows :—

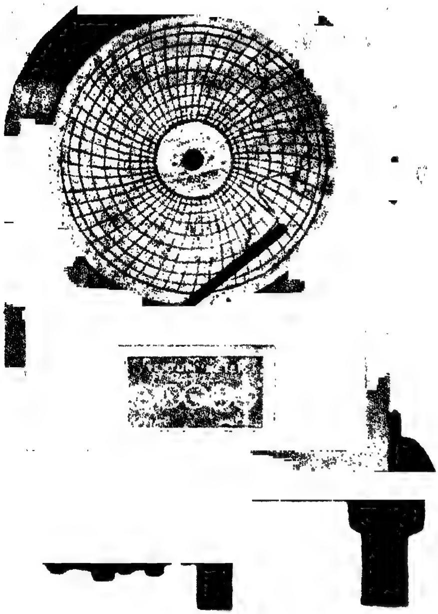


Fig.37—Venturi Meter recorder.

" 'The upper and usually harder formation, through which a well is drilled, is penetrated by a rotary drill to the top of the water-bearing formation. The drill is then withdrawn and the pit casing and an inner casing, to which a coarse screen with an open bottom is attached, are inserted in the hole. The open bottom end of the screen and of the pit casing rest on the water-bearing stratum. The annular space between the two casings is filled with gravel. Sand is then bailed from the inside of the screen which causes it to sink and causes the gravel to lower into the excavation around the side of the screen as it sinks. More lengths of screen or inner casing and gravel are added as the construction of the well progresses. Conditions are finally left as shown in Fig. 38."

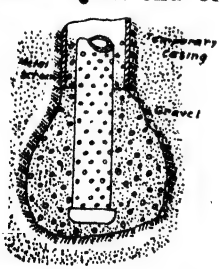


Fig. 38—Shrouding of tube wells.

Filtration Galleries—When the ground flow can be reached at a moderate depth, it is intercepted by infiltration galleries made of wood, brick or stone in the shape of covered drain with openings in the sides and bottoms for the inflow of water. This inflow depends largely on the area of shore surface, through which the water tends to flow into the gallery, and the cleanliness and porousness of the surface. Uniform sized sand grains or gravel offer greater percolating facilities than a mixed variety of coarse and fine sand. The larger the grains, the more the interstices and larger the flow. If the interstices in the layer is small, water will percolate slowly, and proportionately larger infiltration area will be required to deliver a given supply. In deciding upon the site of a gallery, this point should be first considered. The galleries are constructed across the line of flow and are for same reason placed across the channel of a river having a large underground flow to intercept. There are instances, however, in which these are constructed along the bank of a stream. The infiltration gallery for the supply of the town of Brookline Mass lies along the bank of Charles River. The bottom is 6 ft. below the lowest water level in the river, and the gallery is 762 ft. long and has a inside cross section of 4′ × 2′. Lyons in France has two covered galleries along the banks of the Rhone ; the first is 16′-6″ wide and 394 ft. long, and the second is 33 ft. wide and 328 ft. long. The combined area of filtration is about 17,200 sq. ft. and yields about 100 gallons per diem per sq. ft. Fig. 39 shows the cross section of the gallery constructed at Suri. The one advantage of laying the gallery parallel to the flow of river and at the gradient of the ground water table is that the gallery can be extended to any length to augment the supply. At the same time, the infiltration head over every foot length of gallery

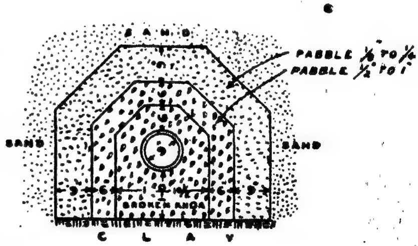


Fig. 39—Section of a Filtration Gallery.

will be the same, and consequently, the velocity of flow will be uniform, and this can be kept down to the critical velocity of sand. Whereas, in the other case, when it is laid across a channel, the infiltration head may not be equal in different lengths of it, and cannot be extended when more supply is wanted. Further, there is considerable risk of its being washed away during a freshet in the rains.

The cost of galleries in this province varies from Rs. 40 to Rs. 80 per foot.

CHAPTER VII

PURIFICATION OF WATER.

Object of purification :—The objects of purification are to make water obtained from natural sources suitable for use both for domestic and manufacturing purposes ; these objects are generally served by removing (1) *bacteria*, (2) *matter in suspension*, (3) *matter in solution*, and (4) *colour*. The most important of these is the treatment of water in such a way as to remove any danger from pathogenic bacteria, thereby avoiding economic losses from death and sickness. The removal of suspended and colouring matter is necessary to make it acceptable to the senses of taste, smell and sight. For industrial purposes, water containing any large amount of suspended or soluble matter is considered entirely unsuitable without purification. It is not difficult to show how an investment made in a pure and wholesome water supply scheme is amply repaid in coin, and also in health and happiness.

Methods of Purification—The methods employed for purification may be briefly classified as follows :—

I. Sedimentation by gravity.

II. Slow sand filtration.

III. Rapid sand or mechanical filtration.

IV. Chemical purification, including softening hard water and removing iron or other objectionable impurities in solution.

V. Sterilization.

It has been found that the method of purification, which is suitable for one kind of water, is quite unsuitable for water from a different source. Sometimes, two or more processes have to be combined to give the desired result. The method of purification to be adopted depends not only upon the physical

and chemical impurities present in the water, but also on the seasonal variation of its constituents.

Sedimentation :—Gravity plays an important part in water purification, as it causes suspended matter heavier than water to settle. When turbid water is allowed to remain quiescent, the heavier portions of the suspended matter settle to the bottom ; but after that action has taken place, sedimentation proceeds slowly so that it is rarely expedient to provide for a longer period of subsidence. Subsidence in 24 hours may reduce the total amount of suspended material by 60 per cent, but it rarely affects the turbidity, the latter being generally due to the fine particles of clay suspended in water. There is a friction between these particles of suspended matter and water which resists subsidence. The greater the ratio between the surface to the weight of a particle, the greater is the friction and the slower the rate of settlement. The subsiding value of spherical particles having the specific gravity of common quartz sand (2·65) has been estimated by the Association of American Waterworks Engineers to be as follows in pure still water at 50°F.

TABLE 32

Dia. of grain. m.m.	Order of magnitude.	Rate of settling m.m. per sec.	Time to settle 1 ft.
10·0	gravel	1000	0·3 second.
1·0	coarse sand	100	3·0 ,,
0·10	fine sand	8	38·0 ,,
0·01	silt	0·154	33·0 minutes.
0·001	Size of bacteria ..	0·00154	55·0 hours.
0·0001	,, of Clay particle ..	0·0000154	230·0 days.
0·00001	,, of Colloidal particle	0·000000154	63·0 years.

It is obvious from these figures that it is practically impossible to remove suspended matter finer than silt by settlement, unless abundant time is allowed to do so. In this connection, it may be mentioned that bacteria, having almost the same specific gravity as water, cannot be removed by the action of gravity, except by being entangled with falling suspended matter.

Another kind of action, however, occurs in settling tanks, viz., a process of fermentation, which takes place in the inorganic matter present in the water and leads to their destruction. In decomposing, the organic matters form carbonic acid and other gases, the ammonia is oxidized and nitric acid, uniting with other salts present, forms nitrates. This process of nitrification is very slow, and to obtain full benefit of it, the storage capacity of sedimentation tanks should be equivalent to a supply of thirty days or more. Sunlight also helps nitrification of organic matters, but it is inoperative beyond the depth of a few inches.

As a general conclusion, it may be mentioned that sedimentation will effect considerable clarification, the extent of which depends on the fineness of suspended matter present in water, but if much of this is fine clay, as generally is the case in this province in the wet season, the resultant water will hardly be suitable for domestic purposes. The water, however, from these tanks becomes more amenable to filtration, and considerably decreases the frequency of scraping or washing of the filter beds, and thereby reduces the cost of maintenance. There are

Fig. 40—Floating arm outlets of Settling Tanks.

generally two types of settling tanks, viz., (1) *intermittent or draw and fill type,* (2) *the continuous type.* The arrangements of the inlet and outlet of the water are such in both types that the water may enter as slowly as practicable at one end of the

tank and may be drawn off in a similar way from the other end without short-circuiting or currents. This is effected by the construction of a cascade at the inlet and drawing the water off by means of a floating arm at the outlet. (Fig. 40).

In the *intermittent system,* the water is pumped or drained, as the case may be, to one of the tanks and allowed to remain quiescent for the period of settlement, and it is then drawn off to the level down to which efficient clarification has taken place, and then the tank is re-filled. In this system, at least three tanks are necessary,—one from which the supply will be drawn, another for filling, and a third for subsidence by complete rest. It may be noted here that in this system for practical purposes settlement does not commence, owing to eddies and currents, until the tank is full and pumping is stopped. On the other hand, water is left much quieter in this system than in the other.

In the *continuous system,* the raw water is allowed to flow at a very slow velocity through one or more tanks. The water during its passage is relieved of its suspended impurities. The fill and draw system is gradually going out of fashion in present day design, owing to the several disadvantages involved in its adoption. A head equal to the depth of the tank is lost, and larger size of tanks are necessary for the same period of settlement owing to the loss of time in filling and emptying. The fill and draw system of sedimentation tanks are more expensive than the continuous flow tanks.

These tanks are generally constructed of brickwork in lime mortar with a floor of brick-on-edge over a layer of lime concrete ; the floor slopes towards a central drain connecting a washout pipe for the removal of the deposit in the bed, the latter being controlled by a sluice valve. The walls have generally an earth-backing from the excavations made in the foundation. Sometimes, these tanks are entirely made of earthen embankment, the portion of which lying below water surface is lined with brick-on-edge over a 6″ layer of lime concrete. These two forms of construction are shewn in Figs. 41 and 42.

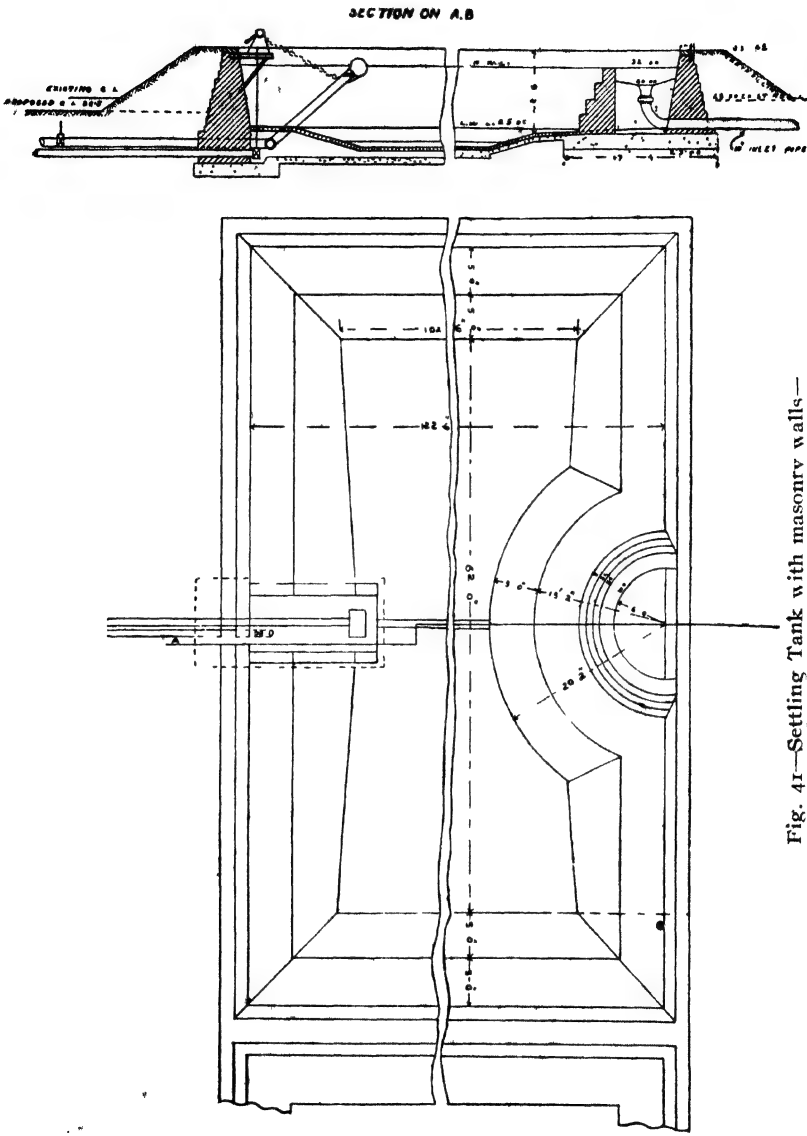


Fig. 41—Settling Tank with masonry walls—Hooghly-Chinsurah Water Works.

In Bengal, these tanks are generally made 8 or 10 feet deep and are used in conjunction with slow sand filters. They usually have sufficient capacity to allow for a rest of 7 to 10 days.

The cost of three settling tanks at Hooghly-Chinsura was Rs. 120/- and at Berhampore Rs. 55/- per 1000 galls.

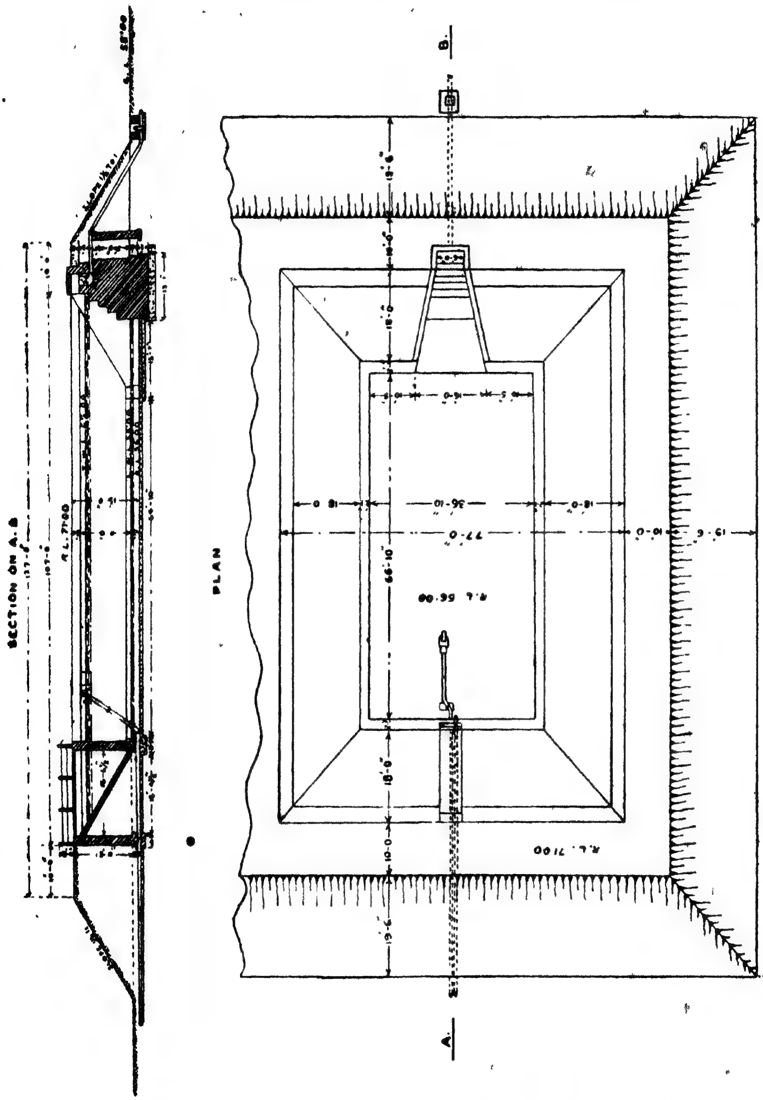


Fig. 42—Settling Tank with earthen embankments—Berhampur Water Works.

Coagulation—Where a reasonable period of sedimentation does not sufficiently clarify the water, so that it can be

purified economically and efficiently by slow or rapid sand filters, it is necessary to resort to coagulation before filtration. In this connection, one term very frequently used is *"turbidity"*. Turbidity and colour are not the same ; the former is due to the presence of clay particles or silt, and the latter may be due to the vegetable or organic mineral present in water. Some years ago the turbidity of water was studied with great care and accuracy by Hazen and Fuller in America. A standard has now been fixed by the U. S. Geological Survey Department and also an instrument has been devised for its measurement (Fig. 43). The turbidity is measured by determining the depth at which the platinum wire one m.m. diameter just disappears when viewed from a point marked near the upper end of the scale. The depth of the wire shewn by graduation will indicate the decree of turbidity. If the water being tested is so turbid that the wire need be immersed until 60 graduation reaches the surface, the turbidity is said to be 60, which means that this is the equivalent to 60 parts of silica per million parts of water in suspension. The silt which chiefly causes turbidity is often composed of particles not more than ·00001 inch in greatest diameter. Very muddy water generally does not contain more than ·1 per cent. of weight of this particle, which will clog any kind of filter so rapidly as to make filtration of the supply on a large scale impracticable.

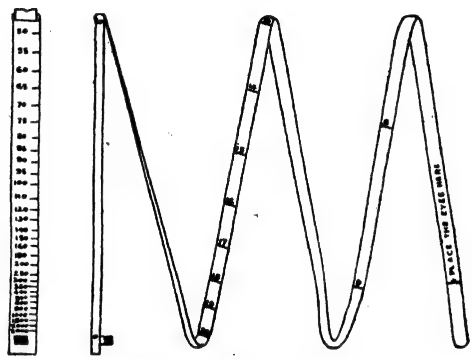


Fig. 43.—Turbidity rod.

Coagulation is the process of adding certain chemicals to water which form, when acting upon alkaline matter present in it, an insoluble gelatinous floculent heavier precipitate, which in its formation and descent through water will absorb and entrain

suspended and colloidal matters, will hasten sedimentation and will remove the fine particles more effectively and rapidly than by simple sedimentation. The addition of a coagulant also removes color, odours and tastes from water. For the purpose of coagulation, aluminium salts are largely used in practice ; they are harmless to consumers when used in proper doses.

Soluble aluminium salts react with carbonates of lime and magnesia in the following typical manner :—

$$Al_2(SO_4)_3{}^1 + 3CaCO_3{}^2 + 3H_2O^3 = Al_2(OH)_6{}^4 + 3CaSO_4{}^5 + 3CO_2{}^6$$

When sulphate of alumina is added to the water, it is decomposed into its component parts, sulphuric acid and alumina ; the former combines with the lime present in the water and forms sulphate of lime, or if enough of it is not present in the water, it remains partly as free acid and partly undecomposed in its original condition. On the other hand, the alumina forms a gelatinous insoluble precipitate, which draws together and entangles the suspended matter present in the water including the bacteria, and allows them to be easily removed by sedimentation or filtration. The practical effect of this reaction is that the carbonic acid in the water is increased, and thereby the water is made more sparkling and piquant, while the carbonate of lime is converted into sulphate ; in other words the temporary hardness is converted into permanent hardness, the total hardness remaining the same, and lastly, the hydrate of alumina, into flocs of coagulant, which are often called "flocs" ; these flocs have a sponge-like structure and have the

1 Sulphate of alumina.
2 Chalk.
3 Water.
4 Hydrate of alumina.
5 Sulphate of lime.
6 Carbonic Acid.

property, as mentioned above, of entangling colouring and other matters in suspension in water.

The amount of coagulant which can be safely used depends upon the alkalinity of raw water. There should always be an excess of alkalinity or lime in the water to be treated. If sufficient quantity of lime is not present in the raw water, the decomposition of the coagulant is not complete ; a portion of the undecomposed coagulant in the form of sulphuric acid, or free alum may pass into the effluent making it unsuitable for domestic supply.

As explained above, the reaction depends for its occurrence on the existence of alkaline matter present in the raw water, and for the purpose of convenience and calculation, the alkalinity is expressed in terms of carbonate of lime present. The quantity of lime in a water available for combination with sulphuric acid can be easily determined by liberation with standard acid with suitable indicator. A method of determining the alkalinity in water with Erythrosen, and determining the quantity of sulphate of alumina and aluminoferric, is given in the appendix III. Although the quantity of coagulant to a given alkalinity can be easily and accurately calculated, it is not safe to use as much sulphate as corresponds to the lime owing to continuous variation in the alkalinity of water during the 24 hours of the day. It is, therefore, considered advisable to use only ¾th. as much sulphate of alumina as is required for the lime present in the water. Theoretically, the addition of one grain per gallon of Aluminium Sulphate to a water naturally alkaline with Calcium and Magnesium Carbonates will reduce the alkalinity when expressed in terms of Calcium Carbonate 7·7 parts per million.

Recent development of the knowledge of Hydrogen-ion concentration has been found of considerable value in controlling the process of coagulation, and consequently filtration, and thereby reducing the cost of operation.

The *Hydrogen-ion concentration* is a measure of an expression of intensity factor of acid and alkaline properties of a

substance as opposed to the quantity factors "acidity" and "alkalinity" as mentioned above. It is represented by the symbol pH, the magnitude of which is expressed on an arbitrary scale similar to that of a thermometer scale, the neutral point of this scale is represented to be 7. The numbers higher than this figure represent alkalinity, the degree of intensity of alkalinity increasing as the numbers increase. Analogously, any value lower than pH 7 denotes acidity, the degree of intensity of acidity increasing as the numbers decrease. A pH value is simply a number denoting the degree of intensity of acidity or alkalinity according to this scale.

From the standpoint of economy of operation and quality of filtrate, coagulation is probably the most important in the water purification. It has been conclusively proved that the most efficient coagulation takes place at a definite pH value which is different for different waters. When alum is used for coagulation, this optimum pH value for the majority of waters lies between pH 5·5 and 7. The optimum may of course change with the season, temperature of water, &c. If at any time, therefore, a good floc is not obtained, the optimum should be redetermined. It has been found, however, that the optimum does not so frequently change, unless the source of supply is extremely variable. Poor coagulation may be caused by an over or an under dose of alum. Once the optimum pH value has been determined for the particular water, the water after coagulation is brought to this figure, either by increasing the amount of coagulant, or the amout of alkali.

Waters that are merely turbid and not coloured can usually be quite satisfactorily coagulated without bringing the pH value down below 7, and consequently are not so liable to require treatment with lime after filtration.

The following table gives approximately the quantity of aluminium sulphate required for different turbidity measured according to the United States Geological Table Standard mentioned above.

TABLE 33

Aluminium sulphate gs. per gallon.	Turbidity.		
	Coarse.	Medium.	Fine.
1·00 ..	300	150	80
1·25 ..	450	200	125
1·50 ..	1000	300	180
1·75 ..	1800	400	225
2·00 ..	...	450	300
2·25 ..	...	600	350
2·50 ..	...	800	400
2·75 ..	...	1000	430
3·00 ..	...	1200	475
3·50 ..	...	1700	650
4·00 ..	...	2500	875

The addition of the coagulants is generally regulated by manual manipulation of the area of an orifice under constant head of the coagulant solution of definite strength. The strength generally varies from 2 to 4 per cent. in practice. The dose of coagulant ordinarily employed ranges from 0·5 to 3 grains per gallon. For a short period to effect high clarification, two or three times this quantity has been used.

The quantity to be used is ordinarily determined by trial and must be such as to produce visible "floc" in the mixing channel, where the water is kept agitated before entering the coagulating basin. The quantity of coagulant should be such as to reduce the turbidity to between 10 to 25 p.p.m. before the water reaches the filters for efficient working. The coagulation tanks are generally constructed in two parts, viz., the *"mixing chamber"* and *"settling chamber"*. In the former chamber, the coagulant is thoroughly mixed by keeping the raw water in constant motion by means of suitably arranged baffleplates, or by paddle wheels or other mechanical devices. Recently in some of the waterworks in India and America, hydraulic jump has been used for this purpose. It is an effective mixing device producing results rapidly and thoroughly. It is possible to design and control a flume in which the jump may be produced and controlled within comparatively narrow limits with

small loss of head. The coagulating tanks are usually of two types, viz., (1) around the end type in which rectangular basin is arranged with baffles, which cause the water to flow in a narrow deep passage with 180 degrees turn at the end (Fig. 44), (2) Up and down type in which the water is made to travel alternately to the surface and bottom by succession of cross baffles as shewn in Fig. 44.

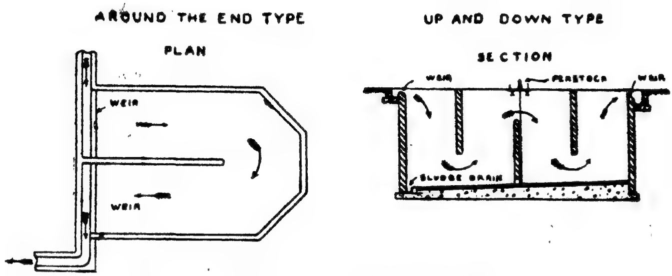


Fig. 44—Types of Coagulating Tanks.

In designing these tanks, which are generally used before mechanical filters, it is customary to allow a velocity of flow of 2·5 feet per minute, and the periods of retention range from 2 to 6 hours, and the longer periods have been found to be of advantage. They are generally made 10 to 15 ft. deep, and have pipe arrangements for removing the sludge intermittently. The arrangement of inlet and outlet pipes are such as to minimise eddies and currents, and short-circuiting as much as possible. The mixing chamber should be of sufficient capacity, so that the raw water may be retained for sufficient length of time for complete reaction, and for increasing the size of the floc. Provided that there is sufficient alkalinity in the water, the reaction with the alumina is practically instantaneous, provided also the alumina solution is thoroughly mixed with the water which takes only a minute or so. To ensure reaction of the alumina with turbid water that requires settlement, a relatively long period of mixing increases the size of the floc and consequently increases the rapidity of settlement. The period of retention varies from 10 to 40 minutes, the longer

period gives better results, but usually 20 to 25 minutes' retention is allowed. These chambers are usually designed on the basis of 1 to 2·5 ft. per minute of linear velocity.

Slow Sand Filtration.

PRINCIPLE :—The most efficient method for a final treatment of water containing finely divided matters in suspension like that in the river waters in Bengal, and one which has proved entirely satisfactory all over the world, is to pass it through a bed of slow sand filter. In this method of purification, water is conducted to the filter beds, which consist of a layer of fine sand (average thickness 2 to 3 ft.) over a layer of coarse sand resting on a layer of gravel supported by a system of suitably arranged brick drains. The use of this type of filter is, however, limited to the purification of water of low turbidity and colour, and comparatively free from excessive sewage pollution. When the turbidity is high, or the colouring matter present is excessive, the water is subjected to some form of pre-treatment, such as sedimentation or coagulation, before it is allowed to pass through filter beds. In the first instance, these filters act as fine strainer ; the interstices in the layer of fine sand being very small, the bacteria and matter in suspension are caught in them forming a film on the surface which in German called "*schmutzdecke*". Franknell and Piefke pointed out in 1887 that until this surface film had formed on the surface of the bed, the filtrate had a very low bacterial efficiency. The film has been described with considerable detail and accuracy by Dr. Keinsua (Trans. of Assoc. of Water-works Engineeis 1899 pp. 40) and others, and is said to consist of a matter of interwoven filaments of algae, bacteria, diatoms and other organised matter together with fine silt. As the film develops but slow at first, the filtrate for the first day or two generally contains a number of bacteria and is inferior in quality. It is, therefore, the general practice in all water-works to run the water to waste until a proper film is formed.

Reinsch by careful experiments at Altona Waterworks has shewn that the matte or slime, which although performs most

of the work, must not be too much relied upon, as considerable number of bacteria is generally found in the sand below the surface film. This observation has been corroborated by the experiments of Van't Hoff. The following table gives the results of their investigations.

TABLE 34

Reinsch.		Van't Hoff.	
Depth below slime in inches.	No. of bacteria as a percentage of those in raw water.	Depth below slime in inches.	No. of bacteria as a percentage of those at the surface.
1·2	12·50	0·0	100·00
2·4	10·30	3·0	5·19
6·3	2·85	7·9	2·12
16·9	1·54	11·8	1·14
		15·8	0·42

Dr. J. C. Thresh has also pointed out that a surface film is not essential for the efficient working of slow sand filters. The efficiency, it is believed, depends upon the formation of *zooglae*, a gelatenous matter containing bacteria, and probably entirely of bacterial origin, round the individual grains of sand in the top layer.

The efficiency of filters depends also on the character of the sand used in beds. The sand must be clean and should consist mainly of sharp quartz grains, the remaining particles may be silicates, though the former is preferable. As regards the physical properties of sand, Allen Hazen carried out elaborate experiments at the Lawrence experiment station, Massachusets, to find out the size of the sand grains most suitable for filters. The results are published in the Report of Mass State Board of Health 1892. A standard has now been fixed to compare the different samples of sand, as in no sample the particles of sand are of uniform size, and are always of

mixed materials. The first relation is expressed by stating the "*effective size*", which is assumed to be the size of the grain such that 10% by weight of particles are smaller and 90% larger than itself. This term is generally used in describing the size of the grains. Another relation is the degree of uniformity of the sand grains ; whether the particles are mainly of the same size, or whether there is a great range in their diameters. This is judged by the "*uniformity co-efficient*", a term used to designate the ratio of the size of the grain, which is 60% of the sample finer than itself. A method of sand analysis, by means of sieves to determine these proportions of a particular sample, is given in the appendix II. At Lawrence, experiments were made to find the effect of the size of sand grains in the percentage of bacteria passing into the effluent, and the results are given in the following table :—

TABLE 35

Effective size of sand grains in millimeters.	Percentage of bacteria passed into effluent.	
	1892	1893
0·38	...	0·16
0·29	...	0·16
0·26	...	0·10
0·20	0·13	0·08
0·14	0·04	0·03
0·09	0·02	0·02

This shows that finer the sand is, the filtrate is more free from bacteria. There is, however, another aspect of the question of the use of very fine sand which is to be considered, viz., scraping. The finer the sand, the more rapidly the filter becomes clogged, and the more frequently it is required to be scraped. The amount of water filtered between two successive scrapings at the Lawrence experiment per acre of filter through various sizes of sand bed is given in the table below :—

TABLE 36

Effective size of sand grains in millimeters.	Water passed between scrapings in Million of gallons.	
	1892	1893
0·38	...	65·8
0·29	...	58·3
0·26	...	47·5
0·20	48·3	...
0·14	37·5	40·1
0·09	20·0	11·7

From this, it will appear that the quantity of water passed between scrapings increases with the increase in the size of sand grains. It is obvious, therefore, that a size of sand must be selected that will give a very good bacterial result without undue expense of scraping and cleaning. Experience has shewn that new sand with an effective size more than 0·35 m.m. will be hardly suitable, and on the other hand, sands finer than ·20 m.m. in effective size will make the working of the filter very expensive. The uniformity co-efficient in European and American filters varies from about 1·5 to 4·0 ; in the majority of filters this coefficient is about 2·0. Sands having a uniform-mity coefficient above 3·0 will be difficult without splitting them up into different sizes. Generally speaking, the more uniform the sand grains the better. The fine sands in Bengal filters are specified to be 0·28 m.m. effective size and of 2·0 uniformity coefficient.

The thickness of fine sands vary considerably in sand filters at different places. In English installations, the depth is from 1 to 5 ft. ; the thickness is seldom allowed to be reduced by scraping to less than 1 ft. The Zurich experiment proved conclusively that fine sand is inefficient when its depth is less than 1 foot, and the German Board of Health, which exercise control over public waterworks in Germany, has limited the

minimum depth of fine sand to 11·8 inches after scraping. The minimum thickness allowed in Bengal is 12 inches.

DEPTH OF WATER OVER FINE SAND—The usual practice is to have a depth of water over the fine sand layer somewhat in excess of the maximum loss of head, so that there can never be any suction in the sand just below the filtering film, and also there may not be undue pressure over the film to damage it. A head of one foot of water over the filter is sufficient to "force water through the filtering matte over a thickness of 1 foot of fine sand." If the film breaks, the water will filter at many times this speed, carrying with it bacteria and other matter present in raw water and also on the sand. To avoid this risk it will be prudent to keep the ratio of the depth of water to the depth of fine sand to unity. The depths of waters actually used in European practice with full depth of fine sand vary from 3 to 4·5 ft. and the latter figure is seldom exceeded. In Bengal filters, the depth is about 2 to 3 feet. At Lawrence experimental filters have been worked with a depth of only 6 to 12 inches without much difficulty.

RATE OF FILTRATION—The rate of filtration in slow sand filters has been different at different times. Formerly, a rate of 6 inches per hour was considered suitable but gradually the rate has been reduced, to increase bacterial efficiency, to 4 inches per hour, that is, about 50 gallons per sq. ft. per day. Franknell and Piefkes' experiments showed that the bacterial efficiency of a filter increases as the rate of filtration decreases.

In Bengal, the rate of filtration usually specified is 3 inches per hour which is equivalent to 37·5 gallons per sq. ft. per day. The area of the filter must be such at this rate as to meet the daily consumption, leaving one filter in case of small and two in the case of large works, out of action for scraping. The number and size of filters required for any particular works are matters more of judgment than computation. They depend upon the arrangement of beds and the cost of retaining walls enclosing the beds.

WORKING AND CONSTRUCTION—The beds may be of any shape or size to suit the exigencies of site. They are usually

shallow open tanks built of concrete or brickworks and rendered water-tight with cement plaster. The construction of a typical filter at Mymensingh is shewn in Fig. 45 in which the details of filtering medium can be found. The bottom of these tanks is generally made to slope towards the outlet and also has a slope towards the main central drain. On the floor and in the bottom layer of gravel, open brick drains are laid to facilitate discharge of the filtered water. The main drain is built of loose bricks laid dry ; the top of this drain is covered with a layer of flag stone, or flat bricks, and the branch drains are laid in a similar way sloping towards the central drain and are usually about 10 feet apart. They have a gradual fall of 1 in 150 towards the outlet. For facilitating the uniform flow of water through the filter bed, S.W. ventilating pipes in connection with the heads of branch darins are built in the longer walls of the filter, the upper end of these pipes being usually finished with short lengths of C.I. pieces with cowls on the top so that undesirable substances may not get into the filtering media.

The water enters the bed through one or more inlet pipes fixed flush with the top of the bed and is admitted at an uniform rate. The uniformity of flow is maintained by keeping a constant head over the inlet pipes by means of float valve fixed inside the inlet chamber (Fig. 45). The water after passing through the different layers of filtering materials is collected in an outlet chamber (Fig. 45) through the under-drains for delivery to the consumers. The water, however, is not allowed to flow into the town main until an efficient filtering film mentioned before has formed ; during this period the filtrate is run to waste by a suitable arrangement of pipes and valves on the main.

It takes about 24 to 48 hours to form a satisfactory film. As soon as the satisfactory filtering layer is formed, the filter is continuously used until the surface is clogged and resistance to the passage of water through the bed is excessive ; a layer of fine sand about half to one inch thick is then scraped off and the filtrate run to waste for the period required for the

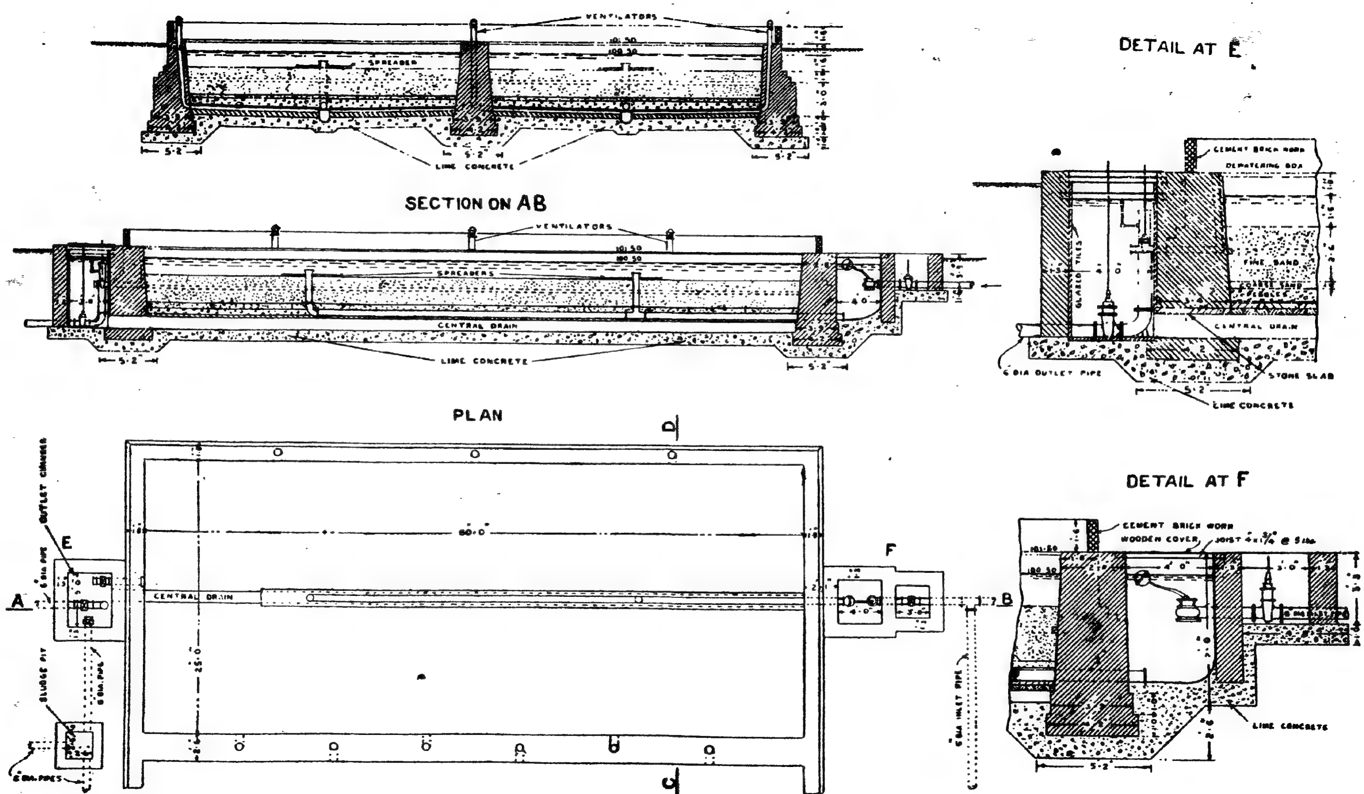


Fig. 45—Slow Sand Filters—Mymensing Water Works.

formation of the filtering layer, and the filter then worked again until clogging necessitates another scraping. Before the thickness of fine sand is reduced to the minimum specified above by repeated scrapings, clean sand is added sufficiently to restore the top layer to the original thickness.

The rate of filtration is, however, maintained uniform by suitable devices to vary the head over the outlet pipe according to resistance, which is equivalent to the difference of the level of water over the bed and that in outlet chamber, and the difference of head is termed *filtration head*. The common form of device in use in Bengal is shewn in Figs. 45 and 46. In Fig. 47 an arrangement for automatic adjustment of the rate of filtration is shewn. The telescopic pipe is carried by floats allowing it to rise and fall with variation of water level, the weir-box having notches or weirs for measuring water discharged. The outflow from the filter can be regulated by means of a small hand-wheel which raises or lowers the weir-box ; once adjusted, the depth of water over the weir remains the same. In the other (Fig. 46), filtered water rises in the outlet

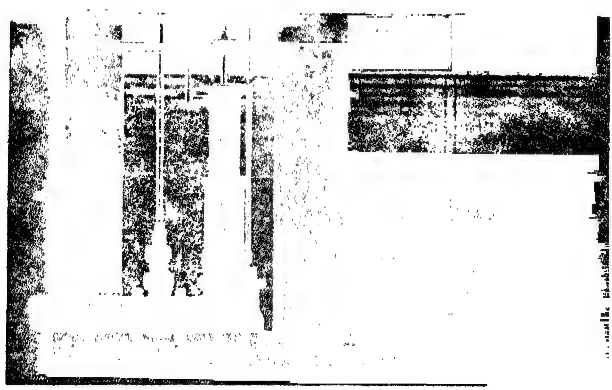


Fig. 46—Adjustable Slow Sand Filter outlet.

chamber and overflows into the bell-mouthed pipe, the filtration head (and thus, the rate of filtration) is controlled by varying the level of the top of the bell-mouthed pipe, which is raised or

lowered by means of a screw working in a sliding nut in the pillar. The best device for measuring filtration head and rate of filtration continuously is afforded by *Peeble's* combination gauge shewn in Fig. 48. The operation of the gauge by means

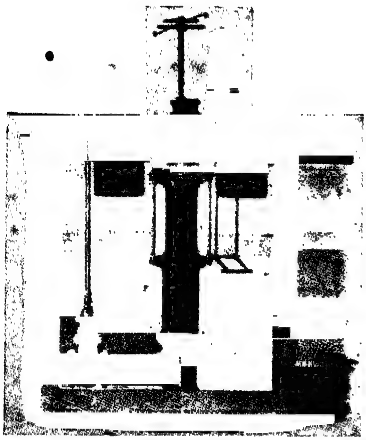


Fig. 47—Slow Sand Filter outlet regulator.

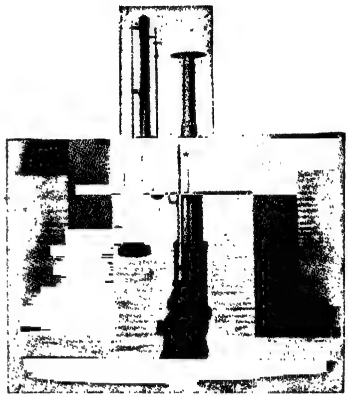


Fig. 48—Peeble's Combination Gauge.

of two floats is very simple and does not require any explanation. This apparatus can be fixed to any existing filter to give complete control over its working. A filter should be started at about one-third of its usual rate, and this rate may be increased gradually according to a schedule until the full rate is reached. Two or three days may be used to bring the filter up to the full rate, but this will depend upon the local conditions.

The chief points to be observed in connection with the management of this type of filter are (1) the thickness of fine sand ; (2) the rate of filtration and the regulation of filtration head ; (3) scraping of filter in proper time and (4) washing sand when used for replenishing.

There are various methods of washing sand ; in case of small works, it is generally found satisfactory to place the sand

on a slightly sloping platform with enclosure on all sides except at the lowest side where weir is provided. The sand is washed with a hose until clear water runs over the weir. For larger works, mechanical washers either of the hopper or drum type are used. At the headworks of Calcutta waterworks, Peeble's sand washer, (Fig. 49) which is found to be more economical than when washing by hand, is used.

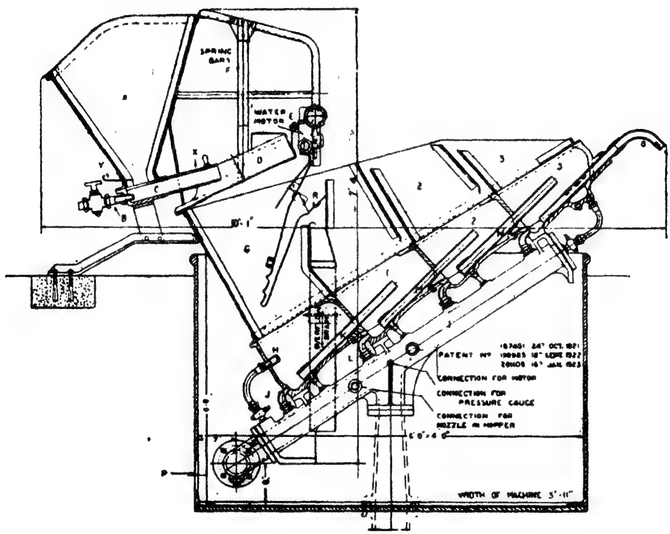


Fig. 49—Peeble's Sand Washer.

This apparatus consists essentially of a series of hoppers and washing chambers and also a silting tank at the bottom. All that is required in the way of power is a low pressure water connection about 3 lbs per sq. inch and the necessary quantity of low pressure water. The dirty sand is filled into the hopper at the top, and from there it is "jetted" to a screen-box which is continuously oscillated by a small water motor. This separates stones, etc. which cannot pass through the mesh but which are washed and "chutted" from the machine.

The sand which passes through the screen falls into the first of a series of washing chambers in which the sand is intimately mixed with the water, which in its upward current carries off dirt and impurities to waste across the overflow.

The sand itself sinks by gravity to the bottom of each chamber from which it is expelled to the adjacent chamber by

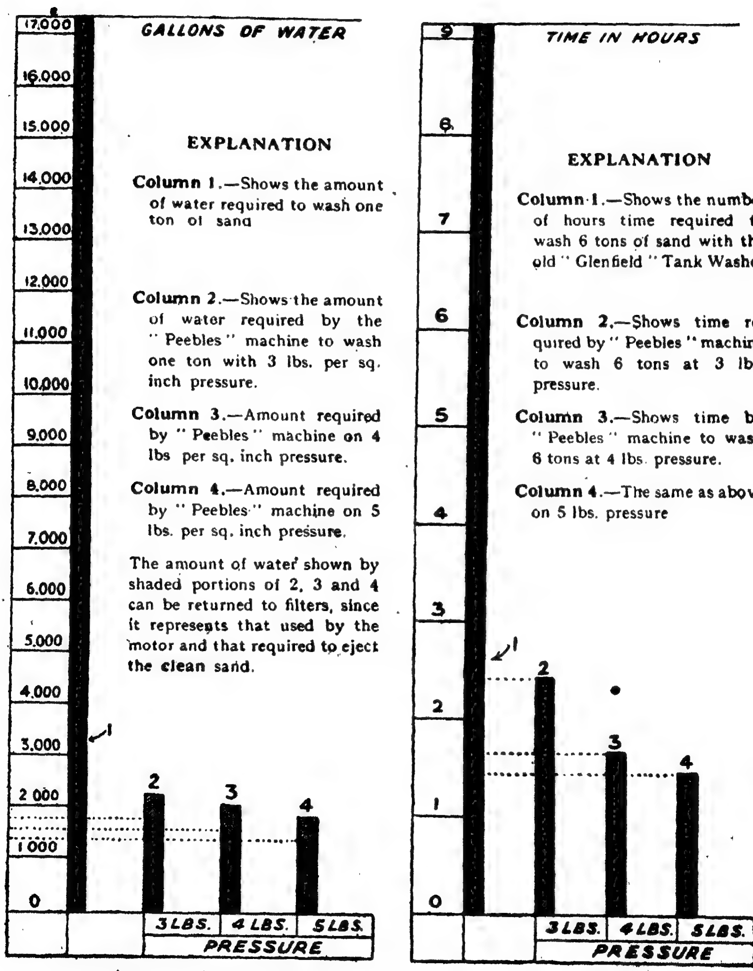


Fig. 50—Comparison of different methods of washing.
(By courtesy of Messrs. Glenfield & Kennedy).

means of jets of water until it is finally ejected from the machine in a stream mixed with water and thoroughly washed of all impurities—no matter what those may be.

The cost of scraping is about five to six annas per 1,000 square feet of filter surface, and that of washing sand by hand is about Rs. 30 per 1,000 cubit feet washed. The cost of annual upkeep of a sand filter is about Rs. 120 per 1,000 square feet of filter surface.

The diagram (Fig. 50) gives an indication of the saving in water and time, resulting in the use of Peebles Sand Washer, as against the best known method of washing by hand and by jets.

Mechanical Filtration—This term is generally applied to a class of filters differing radically both in the rate of filtration and in the method of working from slow sand filters. They were first built in America and consisted of iron cylinders filled with sand through which the water was passed at rates varying from 50 to 100 times the rates adopted for sand. Various improvements have been made since then in connection with the addition of coagulants, washing of beds when clogged, and controlling the flow of water filtered through them ; and their use has now been extended to public water supplies all over the world with considerable success.

Classification —These filters are of two types ; *viz.*, (i) the open or gravity filter (Fig. 51) and (ii) the closed or pressure filter (Fig. 52). In the former, the filtering surface is open to inspection and consequently the efficiency of washing can be watched at all times. In this type, an uniform depth of water is maintained on the filter bed unaffected by the variation of head on the water main, and the chance of breaking the filtering film is minimum. While the latter has the advantage of being fixed direct on the supply main, thus saving the cost of a second set of pumping plant, which is required in the other case. Lately, pressure filters have been used with arrangements to filter under a constant head of water. Fig. 53 shows such an arrangement.

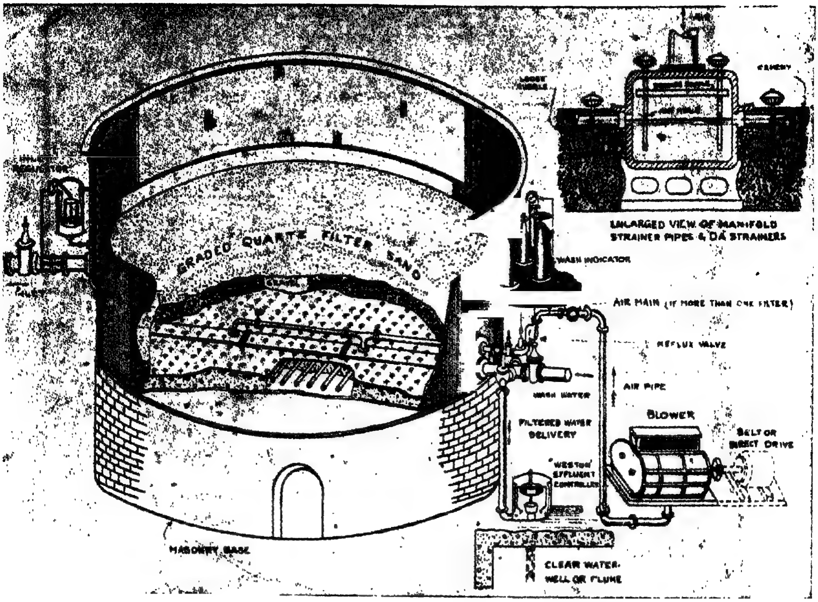


Fig. 51—Mechanical Gravity filter.
(Jewell).

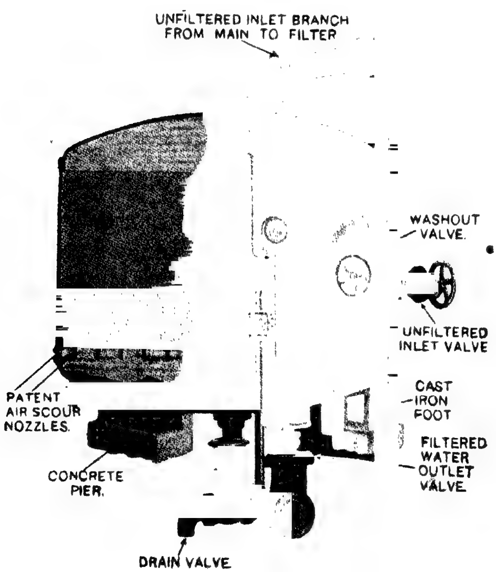


Fig. 52. Mechanical Pressure Filter.
(Candy)

A comparison of Slow Sand and Mechanical filters— Mechanical filters differ from slow sand filters in the following essential points:—

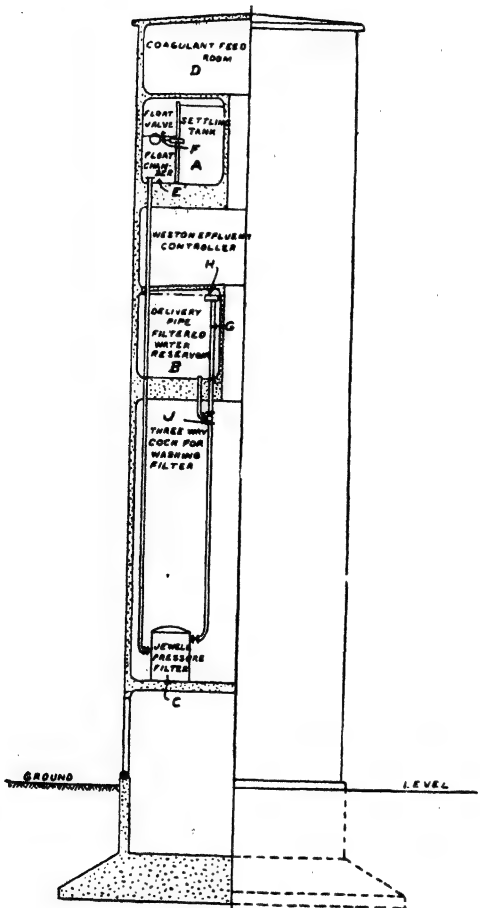


Fig. 53—Pressure Filter working on Gravity System.

(1) The use of more coarse and uniform sand for filtering medium.

(2) The comparative smaller depth of filtering medium usually not exceeding 3 feet.

(3) The use of a coagulating chemical for the formation of an artificial filtering layer, instead of the slowly forming natural slime.

(4) Higher rate of filtration, which is generally 60 to 70 times the rate of slow sand filters.

(5) Cleansing by passing a stream of water upward through the sand with or without mechanical contrivance for agitation, instead of scraping off the surface layer in the case of slow sand filters.

The advantages of mechanical filters are (*i*) low initial cost, (*ii*) smaller area of occupation, (*iii*), a small quantity of filtering

material to handle, and (*iv*) smaller time the filter is thrown out of service for cleansing and washing. Whereas, the slow sand filters are less liable to go wrong even in the hands of careless operators, and do not require any careful attendance, the cost of operation and maintenance of slow sand filters is also considerably less than that of mechanical filters, especially where the water is not very turbid.

PRINCIPLES OF ACTION AND CONSTRUCTION—Since in the case of slow sand filters, the workable filtering matte is formed very slowly, a chemical coagulant is added in this type of filter to produce an artificial *plankton** and artificial filter layer in the place of natural matte for the support of which a bed of coarse sand and crushed quartz is provided, which also acts as a sieve screening the flocs formed by the action of coagulant on the alkaline matter present in the raw water.

By increasing the thickness of the artificial layer, the filtering power can be increased without affecting the efficiency and it has been found to filter water at 60 to 70 times the rate considered suitable in case of slow sand filters. Since the amount of suspended matter retained on the filtering surface is proportional to the quantity of water filtered, it follows that the clogging of these filters should be 60 to 70 times more rapid than in the case of slow sand filters. Moreover, the high velocities, with which these filters are worked, carry the coagulated matter into greater depth of the filtering medium than that in the case of slow sand filters. Consequently, these filters must necessarily be cleansed at much shorter intervals, and the whole filtering medium must be thoroughly washed instead of the top layer only, as in the case of slow sand filters.

Broadly speaking, these filters are composed of a tank containing (1) a system of underdrains at the bottom for collecting the water filtered without loss of sand and for uniformly

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* A term usually applied to the swimming or drifting fauna of the sea.

distributing wash water during the time of reverse flow for cleansing the bed ; (2) a thick layer of filtering medium consisting of more or less uniform particles of quartz sand with an artificially formed matte on the surface ; (3) suitable arrangements for adding coagulant in right proportion to the water to be filtered ; (4) suitable arrangements for draining off the waste water after the medium is washed ; and (5) suitable arrangements of pipes and valves for controlling rate of filtration during working and for facilitating washing when the filter is clogged.

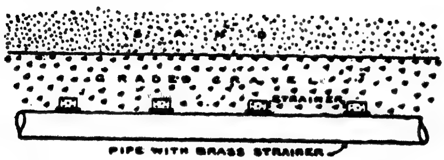

Fig. 54—Pipe Grid with Brass Strainer.

UNDERDRAINS—The following three types of underdrains are in use in rapid filters in this country :—

Pipe grid with brass strainers screwed into the top of the pipe and into branches. (Fig. 54).

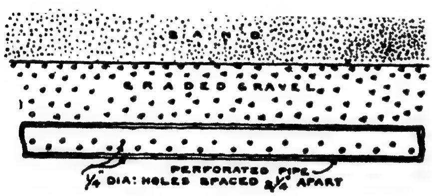

Fig. 55—Perforated Pipe.

Pipe grid without brass strainer ; the underside of the pipes are perforated in one or more rows. (Fig. 55).

Wheeler bottom having marble or cement balls geometrically placed in pyramidal hoppers. (Fig. 56).

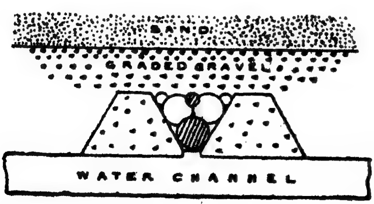

Fig. 56—Strainer-wheeler bottom.

The Candy strainer system consisting of earthenware pipes embedded in concrete in which the Candy patent nozzle is fixed. This system is notable by the fact that there is no iron contained in it and that it embodies separate orifices for the distribution of air. (Fig. 57).

Fine strainers frequently get clogged and have now generally been abandoned. Usually, strainers have coarse aperture and covered with graded gravel 3″ to 12″ thick.

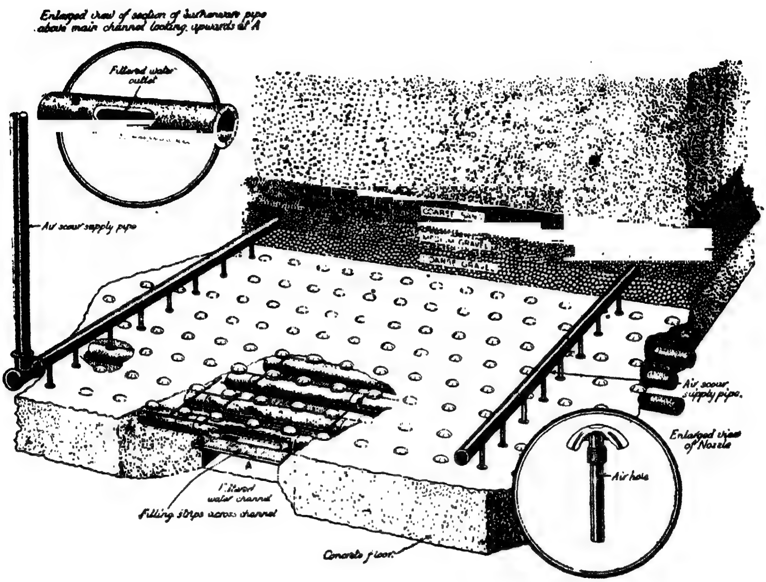

Fig. 57—Candy's Gravity Filter.

SIZE AND THICKNESS OF SAND—Ordinarily, the depth of sand in these filters varies from 2 to 3 ft. but good results have been obtained elsewhere with lesser depth. The sand should be as far as possible pure quartz sand with no acid soluble material. For the facility of washing and packing, each particle of sand used should be as far as possible round. The sand used in these filters has an effective size of 0·30 to 0·50 mm. and an uniformity co-efficient of 1·60. The mechanical filters must not contain any appreciable proportion of fine sand which is likely to be removed by washing.

RATE OF FILTRATION :—These are generally designed on the basis of about 70 to 100 gallons per square foot per hour, and an allowance of 5% or more is made for wash water.

WASHING FILTERS :—All mechanical filters are washed by shutting off the filters, lowering the level of water almost to

the sand, and passing a reversed flow of water up through the sand. There is considerable difference of opinion among waterworks engineers and filter makers as to the most efficient method of washing filters.

In America, the most commonly accepted idea is that it is best to rely on washing with water only ; this system is known as the *high velocity wash.* It consists of washing at the rate of 15 gallons per square foot per minute and upwards and with a head of 30 ft. over the filter bed. The sand is expanded by the upward wash sometimes as much as 50 per cent. This expansion causes the sand grains to rub against one another and loosens the dirt. The washwater troughs must of course be placed at a sufficient height so that the sand will not be washed over by them, and it is usual to make the troughs of sufficient area to ensure that the upward velocity of the washwater will be still further increased as soon as the sand is floated up between the troughs. The design of the whole plant is carefully worked out so that the sand will be floated upto within about three inches of the lip of the trough so as to reduce the volume of dirty water left on the top of the sand at the conclusion of the wash. It is generally laid down that the washwater should not be allowed to travel more than 3½ feet horizontally but this appears to be only another way of saying that troughs should be sufficiently close together to obtain the required increase in upward velocity.

In England, almost all filter plants rely on some auxiliary means of agitation of the sand. This is now usually compressed air, and in older days this was often the mechanical rakes, but the latter have almost entirely fallen out of use.

With sufficient agitation of the sand with the compressed air before washing, the dirt can be removed with a much lower rate of wash, the normal being 5 to 6 gallons per square foot per minute. In many English filters, the system of washwater draw-off is generally similar to the American, but as the *low Velocity Wash* hardly expands the sand at all, there is commonly a very considerable volume of dirty water above the sand when washing. The presence of this water makes it more difficult to

lift the dirt from the level of the sand to the lip of the trough and further leaves a considerable quantity of dirt on the sand at the conclusion of the wash.

There is one English system (Candy System) which appears to be radically different from all others. This firm maintains that it is immaterial how far the washwater is allowed to travel horizontally after it has left the sand and consequently provides no washwater troughs other than small bay at the front of the filter, the sill of which is only three inches above the sand level. From this design it appears that they obtain the same result as the high velocity wash, in that the level of the sand is close upto the sill of the trough when washing, and they do this with a much smaller consumption of washwater.

The quantity of washwater used ranges from 2 per cent to 3 per cent in the case of most American filters, and is about 1 per cent in the case of best makes of English filters.

With the *High Velocity Wash* the gravel at the bottom of the filter has to start with about 2″ pebbles, but with the low velocity wash, the gravel need not be larger than ½ inch.

Filters are usually cleaned when the loss of head reaches 10 feet, but owing to local conditions many filters have been installed in which loss of head can only reach 6 feet. In these cases, washing must be more frequent and the consumption of washwater will necessarily be higher. It is desirable to provide for 10 feet, if possible.

Duration of washing is about 5 to 10 minutes, and in the case of most filters, the time they are thrown out of action after washing is about 15 minutes. During this time the water is run to waste and in calculating the total wastage of washwater the quantity thus used should be taken into account. In the Candy system, no water is run to waste after washing, but the filter is automatically brought slowly into action by an ingenious piece of mechanism.

RATE CONTROLLER :—The efficiency of these filters depends upon the thorough mixing of coagulant in right proportion and upon the filtration being carried out at a uniform rate. Owing

to the rapid variation in loss of head in these filters, automatic arrangements for regulating the rate of flow are necessary ; the arrangement must be as sensitive as possible, so that it can work with a minimum difference of head. For the purpose of regulating rate of flow, three types of controllers are usually used, viz. (1) the same arrangement as in the case of inlet to sand filters, (2) velocity type ; an instance of this type is what is known as "Weston Controller" shewn in Fig. 58. This type

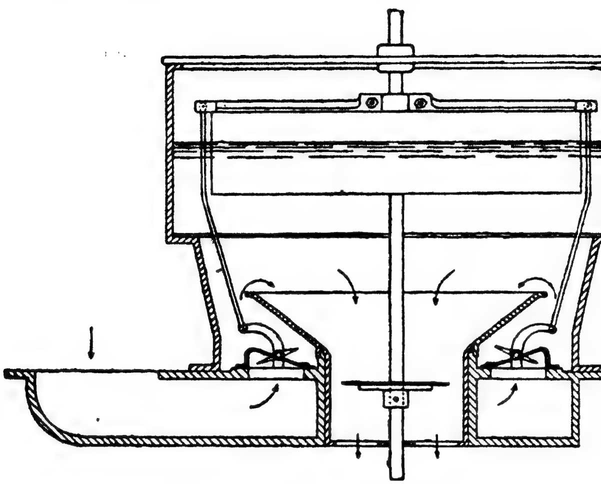

Fig. 58—Weston Controller.

of controller is used in Jewell Filters. The Weston Controller is designed on the principle of ball-cock and orifice. As the float tends to rise with the increase of flow, the butterfly valves to which the float is attached tend to close, thus maintaining a uniform head over the orifice. The type of controller used in the Paterson filter is shewn in Fig. 59. (3) Venturi type ; this is used in larger filters. Fig. 60 shows a controller of this type. This essentially consists of a venturi tube—a valve worked up and down by means of a piston, and a pipe connecting the venturi tube and the top portion of the cylinder in which the piston works. The position of the piston in the cylinder is fixed by the difference between the direct upward pressure on the top of the valve, and the pressure of the venturi pipe trans-

mitted through connecting pipe to the top of the cylinder. The rate of flow through the valve can be regulated by adjusting

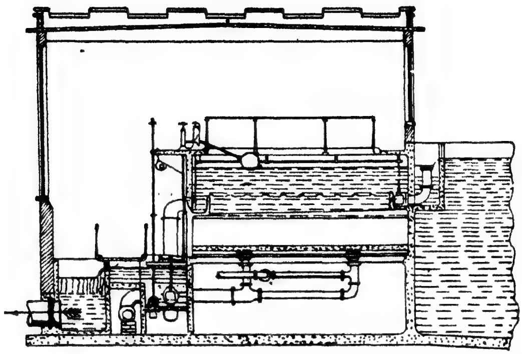

Fig. 59—Paterson Gravity Filter with inlet and outlet.

the position of the counterweight on the lever. (4) The Candy module, which is a type of rate controller used in Candy filters,

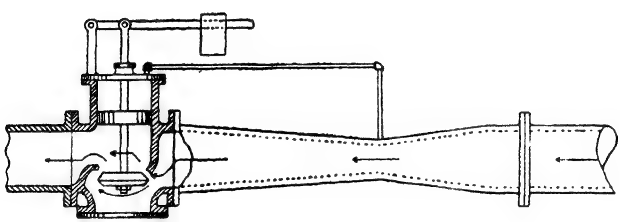

Fig. 60—Venturi Controller.

is shewn in Fig. 61. The water passes through the gap between the piston cylinder and the divisional plate. When the flow reaches the amount for which the module is set, the loss of head through the gap causes a pressure difference on the piston which is just sufficient to lift the piston and valve cylinder, and consequently throttles down any surplus head. The gap is adjustable by means of handwheel at the top of the module and a graduated scale is fitted to show the rate of flow. The operating handwheel and scale are often fixed on the upper gangway and connected to the module by an extended spindle. These

modules are also made of the enclosed type for use with pressure filters.

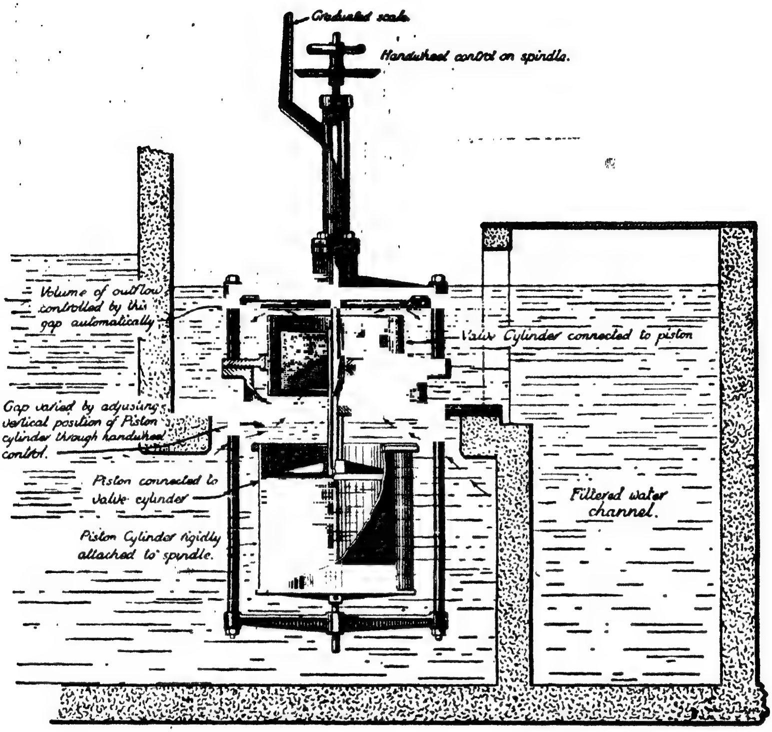

Fig. 61—Candy's Open Type Module.

MIXING COAGULANT—There are several devices for mixing and regulating the quantity of coagulant. The principle in all cases are identical. The coagulant is first diluted to a known strength (usually 2 to 5 per cent) in a tank generally called a mixing tank, then delivered to a second tank to be passed through notch orifice or pipe under a constant head which is maintained by means of a float fixed on pipe feeding the second tank. Fig. 62 shews the proportioning device used in Candy filters. The rate of flow of the coagulant is automatically proportioned to the rate of flow of water to be treated by an interesting device. The coagulant flows over a miniature weir,

the level of which varies proportionately with the level of the water over the main weir. The miniature weir is raised or

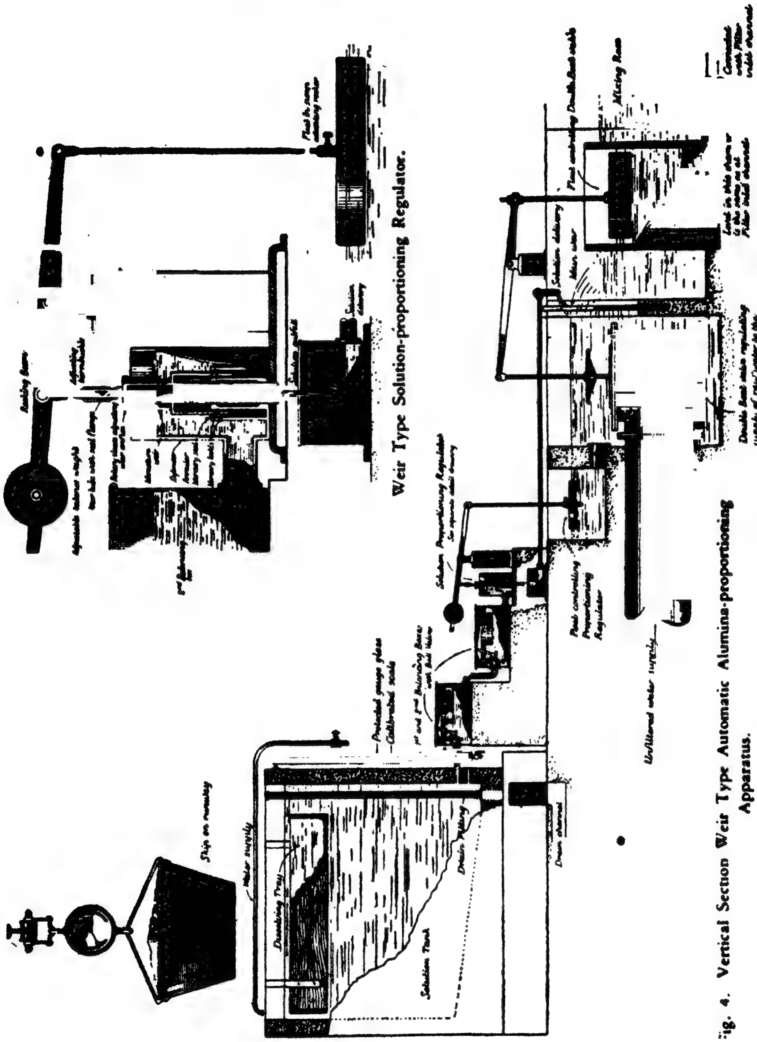

Fig. 62—Vertical Section Weir Type Automatic Alumina-proportioning Apparatus.

lowered by a float. Fig. 63 shows the arrangement which is generally used in Bengal for mixing chemicals in settling tanks in connection with the slow sand filters. Fig. 64 shows the mixing device in connection with the Paterson filters.

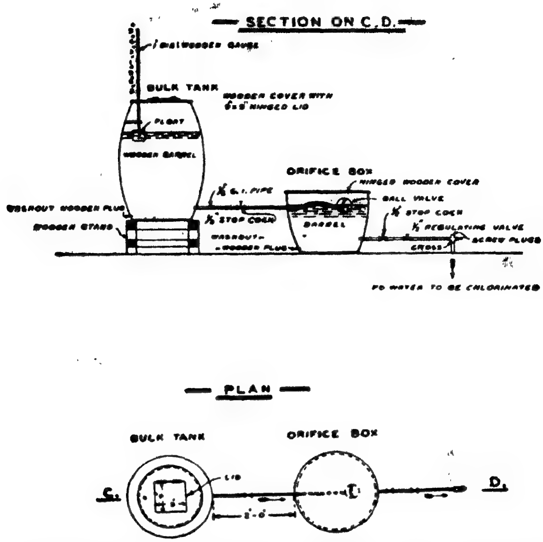

Fig. 63—Arrangement for mixing Coagulant for Chlorinating Water.

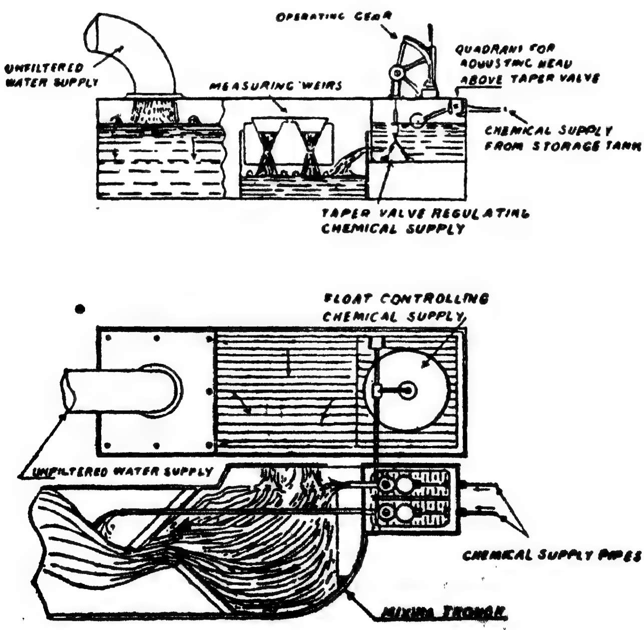

Fig. 64—Patterson Mixing Device.

General working—For the efficient working of mechanical filters, the following points should be carefully observed :—

1. Completeness of coagulation of water to be treated.
2. Proper regulation of the rate of filtration.
3. Careful and efficient washing of filters.
4. Proper watch over the condition of the filter appertenances, such as loss of head, rate of flow, gauges, controller and valves etc.
5. Careful examination of the condition in which the filtering medium is maintained. It should be observed every day, whether any "mud-balls" are forming on the bed ; if so, they must be broken by hand and the sands obtained therefrom are carefully washed and put in position.

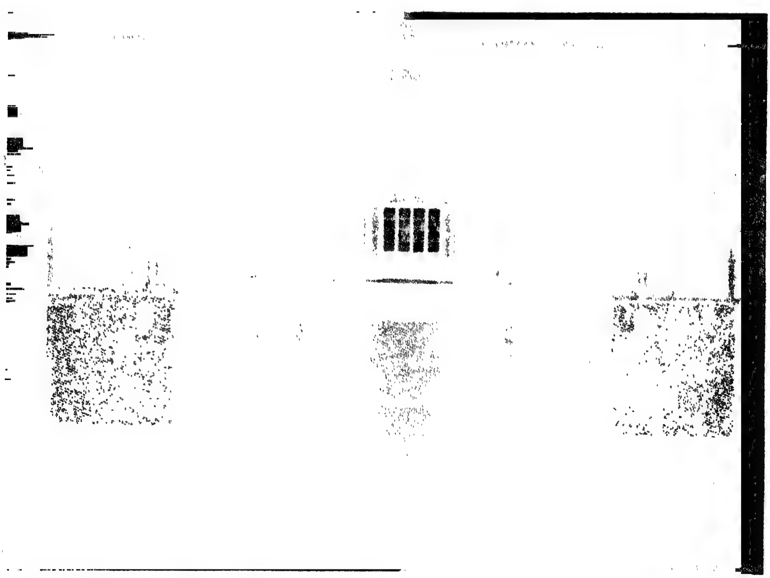

Fig. 65—Operating Gallery—Bangalore Filtration Plant.

Recently a battery of six filters with most upto date control of operation has been erected for the New Bangalore Water Works at Thippagondanhalli, Bangalore, South India.

The filter plant consists of six, one million gallons per day Jewel Air-Wash Rapid Filter units, in rectangular concrete shells. All the valves are operated by haydraulic power from marble operating tables, on which are mounted gauges showing the rate of flow and loss of head in each bed. Each bed is fitted with a venturi type effluent controller, adjustable for rates of flow from zero to 20% overload, with slow start and no-flow-cut-off attachments. Washing is entirely automatic; on closing the inlet valve, the other valves operate automatically and start the air-blower when ready. This is the first plant of its kind in the world to have such automatic control. The photograph (Fig. 65) illustrates the operating gallery of the filter plant.

Chemical Treatment.

Softening of Water—As previously explained, the hardness of water is due to the presence of the salts of calcium and magnesium in solution in water. The salts are generally present in two forms, *viz.*, (1) as bicarbonates, which are termed temporary hardness and (2) as salts of mineral acids, which are called permanent hardness.

Temporary Hardness—The solubility of calcium and magnesium depends upon the quantity of carbonic acid present in solution in water. The free and loosely bound carbonic acid can be easily removed by boiling and can also be removed by adding caustic lime to water. The use of lime for removing the free and half bound carbonic acid, and thereby coverting the bicarbonates into insoluble salts, is the basis of one of the oldest and most efficient process known. This was patented by Dr. Clark of Aberdeen in 1841 and is known as Clark's process. The chemical reaction involved is as follows:—

$$Ca(HCO_3)_2{}^{1} + Ca(OH)_2{}^{2} = 2\ CaCO_3{}^{3} + 2\ H_2O^{4}$$

1 Calcium bicarbonate.
2 Slaked lime.
3 Calcium carbonate.
4 Water.

This equation represents that for every 100 parts of calcium bicarbonate, or for every 44 parts of carbonic acid gas remaining in solution in water, 56 parts of freshly burnt unslaked lime are required and 200 parts of Chalk (calcium carbonate) is precipitated. By this reaction, all the bicarbonate of lime held in solution is thrown down as carbonate with exception of about 2 grs. per gallon which remains dissolved.

With magnesium carbonate, a similar reaction is presumed to take place thus:—

$$Mg(HCO_3)_2{}^1 + Ca(OH)_2{}^2 = Mg\ CO_3{}^3 + CaCO_3{}^4 + 2\ H_2O^5$$

From this, it is evident that 146 parts of magnesium bicarbonate, combined with 74 parts of slaked lime, produces 84 parts of magnesium carbonate and 100 parts of chalk. As a result of this reaction, magnesia is thrown down with the exception of a small quantity remaining adhescent to the carbonate of lime. Magnesium carbonate is soluble in cold water to the extent of 28 gr. per gallon. With the excess of lime, magnesium hydrate which is less soluble may be precipitated.

Consequently, by treating water in this fashion permanent hardness plus 2 grains per gallon of $CaCO_3$ and a good portion of magnesia is retained in solution. During precipitation, however, a portion of the heavy metals, such as iron, magnesia etc. and much of organic matter and clay are carried down effecting improvement as regards potability and appearance to the fluid treated. Precipitation in tanks, however, has got one disadvantage, *viz.*, owing to the process of subsidence being slow, the insoluble matter formed by action of chemical matters may be dissolved again by the absorption of carbonic acid from the atmosphere. Hence, it is better to pass the water after the period of complete reaction through a battery of filters of the type of mechanical filters, which can be readily washed and cleansed as required.

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[1] Magnesium bicarbonate. [2] Slaked lime.

[3] Mag. carbonate. [4] Calcium carbonate.

[5] Water.

Permanent Hardness—The removal of permanent hardness is necessary, especially for water used in boilers and in numerous industries where soft water is required for the successful operation of various processes of manufacture. The removal is also useful in domestic service, as it saves a considerable amount of soap required for the community and gives better facility for cooking. At present, two methods are usually employed in this country in removing hardness of water, *viz.*, (*a*) Lime soda process, and (*b*) Base exchange process.

LIME SODA PROCESS—The chemistry of this process may be probably best understood by knowing the reactions that take place by the addition of lime and soda to water containing calcium and magnesium salts.

Reactions produced by the addition of lime are represented in the following equations :—

(*i*) $Ca(HCO_3)_2$[1] $+ Ca(OH)_2$[2] $= 2CaCO_3$[3] $+ 2H_2O$[4]

(*ii*) $Mg(HCO_3)_2$[5] $+ Ca(OH)_2 = CaCO_3 + MgCO_3$[6] $+ 2H_2O$

(*iii*) $MgCO_3 + Ca(OH)_2 = Mg(OH)_2$[7] $+ CaCO_3$

(*iv*) $MgSO_4$[8] $+ Ca(OH)_2 = Mg(OH)_2 + CaSO_4$[9]

When sodium carbonate is added, the following reactions take place :—

(*i*) $CaSO_4 + Na_2CO_3$[10] $= CaCO_3 + Na_2SO_4$[11].

(*ii*) $CaCl_2$[12] $+ Na_2CO_3 = CaCO_3 + 2NaCl$[13]

(*iii*) $Ca(NO_3)_2$[14] $+ Na_2CO_3 = CaCO_3 + 2NaNO_3$[15]

(*iv*) $MgCl_2$[16] $+ Na_2CO_3 = MgCO_3 + 2NaCl$

(*v*) $MgSO_4 + Na_2CO_3 + Ca(OH)_2 = CaCO_3 + Mg(OH)_2 + Na_2SO_4$.

(*vi*) $Mg(NO_3)_2$[17] $+ Na_2CO_3 = MgCO_3 + 2NaNO_3$.

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[1] Calcium Bicarbonate.
[2] Calcium Hydroxide.
[3] Calcium Carbonate.
[4] Water.
[5] Magnesium Bicarbonate.
[6] Magnesium Carbonate.
[7] Magnesium Hydroxide.
[8] Magnesium Sulphate.
[9] Calcium Sulphate.
[10] Sodium Carbonate.
[11] Sodium Sulphate.
[12] Calcium Chloride.
[13] Sodium Chloride.
[14] Calcium Nitrate.
[15] Sodium Nitrate.
[16] Magnesium Chloride.
[17] Magnesium Nitrate.

By these chemical reactions, the sulphate, chloride and nitrates of calcium and magnesium are converted into sulphate, chloride and nitrate of sodium. By this treatment the soluble salts are not reduced, but in place of objectionable salts of calcium and magnesium, less objectionable salts of sodium are formed, the total remaining the same. The quantity theoretically required for waters of different degrees of hardness can be easily calculated from the above equations. From various investigations, however, the amount of lime to be added thus theoretically proves too high in practice.

The following rules for the addition of lime and soda ash required to soften hard water are adopted by the American Waterworks Association.

Lime required (pounds CaO per 1000 galls.) to remove carbonate hardness is reckoned as follows:—

1. The sum of free and half bound CO_2 (expressed parts per 1000,000) multiplied by 0·0127=pounds of CaO per 1000 gallons required to absorb the free and half bound CO_2.
2. Total magnesium (expressed as parts per 1000,000) multiplied by 0·023=pounds per 1000 gallons of CaO necessary to precipitate the magnesium.

Soda ash required (pounds of Na_2CO_3 per 1000 gallons) to remove non-carbonate hardness is computed as follows:—

Non-carbonate hardness (expressed in parts per million) multiplied by 0·0108=pounds per 1000 gallons of Na_2CO_3 (sodium carbonate) required to be added.

The *Lime Soda Process* is perhaps the oldest and the most widely applied process known for softening. With this process the hardness cannot be entirely removed, but a residual hardness of about 3° to 4° still remains in the water treated. For all practical purposes, this water is considered to be sufficiently soft.

The softening of large volumes of water with chemical reagents, necessitates the thorough mixing of the requisite quantity of the reagents with water, the requisite period of time for complete reaction and suitable arrangements for the efficient

removal of the products of reaction. There are many types of water-softeners in the market, but a mechanical filter plant can be adopted with slight modification to do this kind of work as efficiently as possible. The principal requirements for this kind of work are reaction or settling tanks of sufficient capacity and suitable arrangements for the proper and efficient control of mixing and regulating the chemicals. The solution tanks must be of larger size than is generally used in ordinary machenical filters. The use of mechanical filters has the advantage of not only softening the water, but also making it bacteriologically pure.

Base exchange process—This is the most recent method of water-softening and is gradually coming into extensive use. Dr. Glans, President of the Laboratory for Geonomy at the Royal Geological Institute, Berlin, introduced a class of rock-minerals, called *zeolites*, which has the peculiar property of removing lime magnesia, manganese or iron from water when slowly filtered through it. These minerals in their natural state consist of hydrated silicates of aluminum and alkalies; Dr. Glans classed them as (i) double silicates of aluminum and (ii) aluminum silicates. The former class of zeolites has little power for the exchange of bases mentioned above, but the latter class is very active. Among the natural zeolites, are *analime, genelinite, sodalite, chabasite* and others, of which *chabasite* is found to be most active.

Natural zeolites are comparatively rare silicates, and this led Dr. Glans to produce artificial zeolites, and the synthetic zeolite, he produced, possessed the same peculiar characteristics of natural zeolites, but in more intensive degree. The artificial zeolites are called *Permutit*. This mineral has a capacity of 30 per cent. in excess of the most active natural zeolite. The approximate composition of Dr. Glans artificial zeolites is $2SiO_2Al_2O_3Na_2O$.

Permutit is manufactured commercially by fusing felspar, kaolin, clay and soda in a furnace, similar to that used in the manufacture of glass. When the material so fused has cooled,

it is crushed and washed to remove the alkalies and sold as *Permutit* in the market in granular form having particles from 0·5 to 2·5 m.m. in diameter. Permutit contains 46 per cent. of silica, 22 per cent. of alumina, 13·6 per cent. sodium oxide and 18·4 per cent. of combined water.

Another synthetic zeolite is *Doucil,* manufactured by Joseph Crossfield & Sons of England. This has the same chemical properties as zeolite, but has a higher exchange value ; that is, it removes more hardness per pound of material used.

Natural or artificial zeolites gradually loose their capacity of softening by exchanging one of its constituents for a base in the solution on which it acts. This process would have hardly any practical value, if it had not been discovered simultaneously that this absorbtive capacity can be regenerated by washing it with a solution of common salt. The nature of reaction between hard water and permutit may be represented as follows :—

(i) $2SiO_2Al_2O_3Na_2O + CaCO_3H_2CO_3 = 2SiO_2Al_2O_3CaO + 2NaHCO_3$

(ii) $2SiO_2Al_2O_3Na_2O + MgSO_4 = 2SiO_2Al_2O_3MgO + Na_2SO_4$.

The reaction due to the addition of sodium chloride (common salt) for regeneration may be shewn thus :—

$$2SiO_2Al_2O_3CaO + 2NaCl = 2SiO_2Al_2O_3Na_2O + CaCl_2.$$

The practical application of this method of softening is usually effected in filters resembling a mechanical gravity or pressure filter, in which the permutit takes place of sand (Fig. 66). Permutit, however, has not any capacity of reducing the bacteria in water or its turbidity, and it is therefore necessary to filter such waters to obtain the best results. The thickness of permutit softeners varies from 2 ft. to 6 ft. in practice. The rate of flow in this class of softeners varies with the depth of mineral and hardness of water to be treated. Wickware, in a paper on "European Power Plant Practice" says that the usual rate of flow through the bed is from 10 to 16 ft. an hour, and it decreases with the increase of hardness.

The quantity of common salt required for regeneration of zeolite varies from about 4 to 8 lbs. per lb. of hardening salts

removed expressed as calcium carbonate. The regeneration is generally done with a 10 per cent. salt solution, which is placed above the filter and is allowed to run slowly through the bed. It takes about 8 or 10 hours and can be done at night, or in case of continuous working, duplicate filters can be provided so that one can be worked while the second is to be regenerated. After regeneration, the filter is washed for half an hour or so to remove the salt and air that may have accumulated.

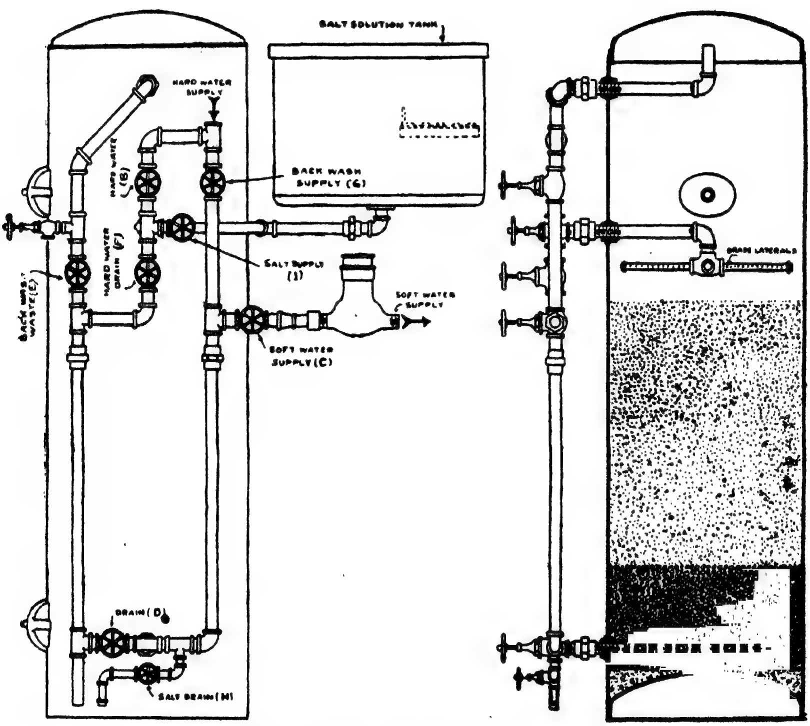

Fig. 66—Base Exchange Softener.
(Jewellite).

Doucil having higher exchange value requires smaller plant than Permutit. With Doucil, the regeneration is practically instanteneous, as soon as the Doucil comes in contact with the salt solution, so that the whole process of regeneration takes about 15 minutes. The amount of common

salt required for regeneration with Doucil is about 2½ to 4½ lbs. of salt per lb. of hardening salt removed, expressed as calcium carbonate.

It will be noticed from the reactions given above that in softening water with lime and soda ash, the residual hardness in treated water is still considerable, while by passing hard water through a permutit filter, the hardness can be reduced to zero. This is the only practical method so far known for the complete softening of hard water. It may be noted, however, that in water treated in this way an equivalent of sodium carbonate is always left. According to Hoover & Scott, the following are the advantages and disadvantages of this process of softening water :—

ADVANTAGES.	DISADVANTAGES.
1. This is the only process by which the hardness can be reduced to zero.	1. Cost of operation is much higher than the lime and soda process.
2. One chemical is required.	2. The water to be treated must be free from turbidity, otherwise the pores of permutit will be clogged.
3. Not affected by the variations in hardness.	3. Water softened in this way contains sodium-bicarbonate, and if used in boilers, may cause "foaming".
4. There is no sludge to be removed.	

This process of softening cannot be used successfully to treat waters with excessive sodium salts, nor with water containing free acid or large amounts of iron, without previous purification.

Another chemical called *alligit* has lately been prepared which, it is claimed, is acid-resistant and unaffected by free carbonic acid in water. It is somewhat similar in compostion to permutit and possesses all its softening properties.

Deferrisation—Owing to the success of tube wells in some parts of the country, the use of ground water for public water supplies is gradually becoming very popular. The waters from this source in majority of cases are not entirely suitable for domestic and industrial purposes without purification. In some cases, they contain an objectionable amount of iron which is required to be removed before being delivered to the consumers.

When water containing iron is freshly drawn from the well, it is generally clear, but gradually becomes turbid owing to the separation of a brown precipitate formed by the action of air on the iron in solution. This turbidity due to iron is not so much objectionable from the hygienic point of view as from that of appearance, and the trouble it causes afterwards. H. Schwers thus sums up the different aspects of the question. "The inconveniences from iron appear sometimes to commence when the content reaches ·1 part of Fe per million ; commonly however, when it rises over ·2 to ·3 part. With ·1 to ·3 parts a deposition of iron hydroxide occurs on standing ; this concentration is found most favourable in the growth of iron bacteria which may block up pipes, and also at this point takes up a chalybeate taste and becomes unsuitable for washing and many industries. The growth of iron bacteria, such as *crenothrix*, has been troublesome at Rotterdam, Berlin and other places. This bacteria develops in darkness and form distribution pipes the most favourable place for their growth, and in many cases they have seriously interfered with the supplies."

Iron is generally present in ferrous state, that is, as ferrous hydrate, ferrous bicarbonate or ferrous sulphate. In some cases iron is held in solution by the organic matter present in the water.

The process of deferrisation, or removal of iron from water, is based on three physico-chemical actions, as described by Dr. J. Tillmans, Director of the Chemical Department of Municipal Hygiene, Germany.

Firstly, by contact of dissolved iron with air, oxygen converts the ferrous compound into insoluble ferric compounds. Consequently, the water may be aerated and subsequently filtered. One part per million of oxygen will oxydise 7 parts of iron, and an excess of oxygen may be obtained by slight exposure to air. The ground water under ordinary conditions of temperature contains about 10 parts per million of dissolved oxygen. Secondly, since carbonic acid keeps the iron in solution, the precipitation of iron may be effected by the ventrallisation of the carbonic acid with lime. And finally, if the iron is present in water in colloidal form (organically combined), a coagulant such as aluminium sulphate or ferric chloride is employed.

The first two methods are generally used in Germany and the last in America. In Europe and America, numerous patents have been taken for deferrisation plant with considerably varying systems of aeration and filtration. The principal methods of aeration adopted in Germany are cascades, raining devices with suitable perforated plates, coke towers, wood shavings, clinkers, and glazed bricks. Through the aerators, the water is allowed to flow at a velocity of 15 to 48 feet per hour ; any velocity above 48 feet will require sedimentation in tank before filtration. Another method of removing iron by aeration and filtration is Candy's method, in which the whole process is done in the filter plant itself without any auxiliary cascading or other means of aeration. This system has been very largely employed in England, notably at the Waltham Abbey Pumping Station of the Metropolitan Water Board, London, and a very large number of other towns.

In the method of aeration through orifices or nozzles, a certain head over the orifice is not allowed to exceed, as by doing so, the small threads of water tend to coalesce making

good aeration impossible. The following heads are usually adopted in practice for the different sizes of orifice:—

TABLE 37

Dia. of Orifice in inches.	Head in feet.	Delivery per hour in gallon.
0·040	0·67	0·87
0·036	0·92	0·83
0·032	1·00	0·67
0·028	1·33	0·52
0·024	1·83	0·52

Similarly, the method of filtration varies very considerably. Generally, they are made of sand and gravel, the size of the particles being very important for the complete retention of ferric hydrate particles. The washing arrangements of filters and aerators are also different in different systems. In some filters, where the medium is entirely of gravel, the size of particles is on the average of about ·20 inch, and the rate of filtration is from 12 to 70 ft. per day. The removal of iron can easily be accomplished in conjunction with common mechanical filters by making suitable arrangements for aeration followed by treatment with lime or aluminium sulphate, or both ; as in some cases, the precipitate formed by the action of lime alone is too fine to be removed by filtration. The addition of aluminium sulphate is specially necessary, when the iron is present in organic state.

The methods of deferrisation described above are intended for large waterworks for municipal or industrial purposes. There remains to be mentioned that which is usually adopted in other countries for the removal of iron from well-water for individual house supplies.

The simplest method of doing so is to put in several buckets of iron-free water at the same time saturated with oxygen by standing in the air. This will make the water drawn from the well free from iron for many days, but this has the disadvantage

of increasing the chance of pollution of supply and choking the strainer.

For this purpose, Dunbar filters have also been used. It consists of an iron cylinder containing sand of 1 to 1·5 millimetres in diameter ; the cylinder is repeatedly emptied and filled for the purpose of air circulation among the interstices of sand grains. It is found that once the sand grains have obtained a coating of ferric hydroxide, the filter prevents any more iron to pass through.

The third device is what is known in Germany as Deseniss and Jacobi system of bastard pump. This differs from all other systems in that that when the water is delivered from the pump, it is already free from iron, thus obviating the necessity of the use of intermediate scrubbers or clarifying reservoirs. The arangements and the method of working of the pump are as follows :—

On the top of the pump cylinder, a second cylinder of double the capacity is attached. Both the cylinders have closely fitting pistons provided with valves and worked by the same piston. The water flowing from the lower cylinder to the one above is mixed with equal volume of air sucked through a valve in the upper cylinder. The water from the upper cylinder is then forced through a filter composed of sand of 0·5 millimetre diameter, and thereby comes out free from hydroxide of iron that has been formed. It is not necessary to renew the filter layer at all ; the cleansing of the medium is effected by a reverse flow arrangement by means of stop cocks.

The methods of deferrisation described above cannot entirely remove iron from water, a residue of ·1 part per million remains after treatment.

Sterilisation.—The removal of pathogenic bacteria, and other organic matters from water by the methods described in the foregoing pages is chiefly accomplished by subsidence for a long period, and by the surface skin in slow sand and rapid filters. In spite of all engineering improvements, these methods give more or less an efficient mechanical straining,

which has on many occasions broken down owing to inefficient working or to accidents beyond human control. The direct application of disinfectants to water used for domestic purposes has on this account appealed to many sanitarians as a prophylactic measure. Such treatment should be considered as an additional safeguard, and under no circumstances, should reliance be placed on it so as to neglect any process of working filters.

Broadly speaking, there are two methods of sterilisation, *viz.*, (1) physical method ; (2) chemical methods. In the first process, heat and light are the chief agencies employed for purification. But this method has been found to be expensive and considered to be impracticable, where large quantity of water is required to be treated.

The application of chemicals to water for this purpose has received considerable attention towards the latter half of the nineteenth century, and halogens and their derivatives, compounds of copper, permanganates, caustic lime and other agents possessing oxidising property have all been used from time to time for sterilising water. The agent now generally employed is chlorine in liquid from, or as hypochlorite of lime or bleaching powder, or as sodium hypochlorite. So far as sterilisation is concerned, the effect mainly depends upon the available chlorine in the chemical applied, and consequently, with regard to efficiency there is little to choose between the forms employed. The "*available chlorine*" is that which is active in oxidising or has the germicidal capacity as indicated by potassium iodide and starch test described hereafter. Of the different compounds of chlorine mentioned above, "bleaching powder" is generally used. Bleaching powder is manufactured by passing chlorine gas over slaked lime, and the reaction leading to its formation may be expressed by the equation $Ca(OH)_2 + Cl_2 = CaOCl_2 + H_2O$ which represents that the chemical thus formed contains about 50 per cent. of chorine, but the commercial products do not give more than 35 to 37 per cent. of chlorine, the average being about 33. In addition to chlorine, it contains about 40 to 45 per cent. of lime, and the remainder

water which remains either in the free state as moisture, or in combination with the chemical. Bleaching powder should be kept dry, as it rapidly deteriorates by the absorption of moisture and carbonic acid when exposed to the atmosphere.

Chlorine gas is almost universally employed for plants of more than 500,000 gallons per day capacity, and sometimes for smaller plants as well. The chlorine is usually supplied in cylinders containing 65 or 70 lbs. of pure chlorine, the whole of which is available for use. Many reliable chlorine gas plants are manufactured by the principal filter-makers.

An entirely new system for the removal of pathogenic germs has recently been introduced under the title of *Catadyn*. In this system, the water is brought into contact with silver pellets which have been tested over a considerable period by many well-known analysts, including Drs. Thresh, Beake and Suckling. In all cases, the reports have been eminently satisfactory. The advantage claimed for this system is that any possible taste trouble due to chlorine is eliminated.

CHEMISTRY OF ACTION:—Hypochlorite of lime has long been in use for lime and other materials, for bleaching and this action was attributed to the nascent oxygen produced in the presence of moisture. Later, when this and other chemicals of allied composition came to be used for deodorisation or disinfection, their action was attributed to the same cause. Earlier experiments of Fisher and Proskaner support this view. They further found that humidity played an important part in this process, as it favoured oxidation. But subsequent experiments of Warouzoff, Winogradoff and others shewed that chlorine has got toxic action on bacteria, as tetanus spores were destroyed by a mixture of chlorine gas and air in one minute.

When this chemical is mixed with water, it undergoes a molecular rearrangement which is expressed in the following equation:—

$$2\,CaOCl_2{}^{1} = Ca(OCl)_2{}^{2} + CaCl_2{}^{3}$$

1 Bleaching Powder.
2 Calcium Oxychlorite.
3 Calcium Chloride.

When Calcium Oxychlorite is applied to raw water, the following action takes place:—

$$Ca(OCl)_2 + CO_2{}^{1} + H_2O = CaCO_3 + 2HClO^{2}.$$

Thus, Hypochlorous Acid is produced.

The hypochlorous acid is unstable and gives up its oxygen to organic matters. Being a weak acid it has no appreciable effect on $CaCO_3$ formed. During decomposition of this acid, a considerable amount of energy in the form of heat is evolved. This occurs whether the acid is present alone or with other substances. In the presence of oxydizable matter, the decomposition takes place very rapidly and the energy of reaction accounts sufficiently for the destruction of impurities in the form of bacteria or other organic cells.

The decomposition of hypochlorous acid may be expressed in the following equation:—

$$2HClO = 2HCl^{3} + O_2 \text{ (Oxygen)}.$$

The time required for the complete reaction to take place is variable, being under most favourable conditions about thirty minutes, but increasing as the temperature of the water to be treated decreases, and also being longer in water containing half bound carbonic acid than in those containing free carbonic acid.

The amount of available chlorine required to sterilize the water depends upon the amount of organic matter present in the water. The following test known as Horrock's test was applied during the Great War in determining the quantity of chemical required for sterilization:—

Two grams of bleaching powder (33 per cent available chlorine) is added to 250 c.cs. of water making a solution free from lumps of lime. The water to be tested is placed in six glasses containing 165 c.cs. One drop of the solution from a pipette (·065 c.c.) is added to the first, two to the second, and so on, giving respectively 1, 2, 3 and 4 parts of free chlorine per

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[1] Carbon Dioxide.
[2] Hypo-chlorous Acid.
[3] Hydrochloric Acid.

million of water. The glasses are then allowed to stand for half an hour for the oxidation of the organic matter present. Then, the water is treated with potassium iodide and starch, commonly known as test solution. The presence of chlorine in excess of what is required for oxidation is indicated by the water giving a dark blue colour. A pale blue colour indicates a dose just enough, while no change of colour indicates insufficient dose of chlorine added. It is always desirable to add chlorine a little in excess. The amount required usually varies from ·3 to 1·5 parts of available chlorine per million.

The bleaching powder is introduced into a solution in a manner similar to coagulants. Apart from providing for the corrosive properties of the solution and for its offensive odour, the tanks and other arrangements may be similar to those employed for solutions of other chemicals that are commonly

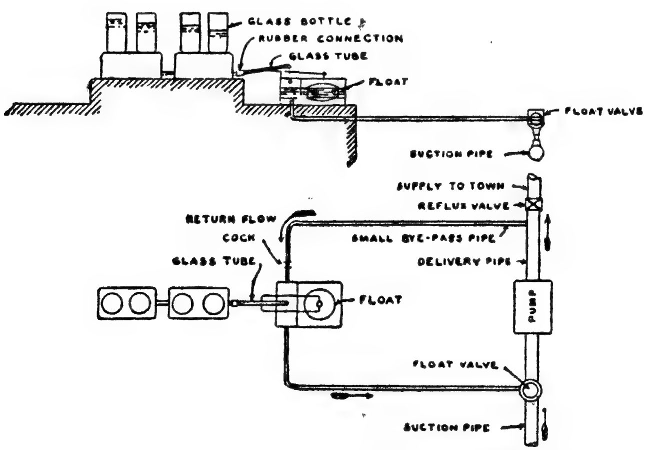


Fig. 67—Arrangement for Chlorinating Water.

prepared and applied to water in purification process. A simple apparatus consisting of a main vessel of glazed stone-ware, two glass bottles, and a glass tube with rubber connections as shewn in Fig. 67 has been patented by Mr. F. C. Griffin, Chief Engineer of the Public Health Department, Bengal, for small water supplies. The bottles are filled with solution when

standing alongside in upright position. They are then turned over in the hands, and inserted on the main vessel. Then, there is a flow through the glass outlet pipe, which can be regulated by raising or lowering the pipe by means of toggle or a regulating screw.

The oxidising action of chlorine being entirely dependent on contact, a thorough mixing with water to be treated is considered essential for the successful sterilization. The greater the volume of solution, containing the chemical compound with the volume of water to be treated, the more easily the mixing will take place. For large plants, a 2 per cent. solution may be used, and for smaller one, a 1 or 0·5 per cent. solution will be easy to handle. In making up the solution, sufficient water should be first added to make it into a thick paste, as bleach does not readily dissolve in water. Half a gallon of water to a pound of powder will be found convenient.

Water, when treated with hypochlorite, has very often a pungent odour and acid taste characteristic of chlorine that renders it offensive to the nose and palate. These tastes and odours are not so much due to the chlorine as to chlorinated organic compounds, formed by interaction with the organic matters present in waters. Consequently, a water may often taste or smell, although the chemical test cannot detect any indication of free chlorine. The taste and odour due to residual chlorine may be eliminated by storage or by application of ammonia and activated carbon ; experiments on a chlorinated water indicated that 1 part per million of available chlorine in water could not be tasted after three hours storage, and this period can be further reduced by aerating the chlorinated water. The period of contact before delivery to the consumer should not be less than half an hour in any case. Another disadvantage in chlorinating water is the corrosion of pipes and pumps. At Uttarpara, where the water is chlorinated throughout the year, the impellers of the centrifugal pump have to be changed very frequently. Galvanised iron pipes are more readily affected than the pipes coated with Dr. Angus Smith's solution. With regard to the continuous use of bleaching

powder, or other chlorine compounds in public water supplies, the following remarks of Dr. J. Tillmans of the Municipal Institute of Hygiene will be of interest.

"In the opinion of the author, it is of advantage to use the method for a time with a certain amount of scepticism, as suspicion always prescribes that to be certain of the destruction of all the bacteria, such an excess of bleach is necessary that it is noticeable in smell and taste or vice versa that if there is no smell and taste in the purified water, the bacteria are not destroyed with certainty.

Add to this also, that it is not yet established that the continuous use of small quantities of bleach, even should they be imperceptible by the senses, is not a matter of serious misgivings.

For this reason, this method cannot be recommended for general imitation. It might possibly be of service in certain circumstances, if there were temporary dangers with drinking water as in times of epidemic, or it might perhaps be used with success in time of war".

On the other hand, many of the water-works in England and America are now a days treated continuously with chlorine as an additional safeguard without any appreciable disadvantage, although in some cases complaints have been heard due to this but no confirmation has been received up to the present time as to the truth or otherwise of these complaints.

CHAPTER VIII

CONVEYANCE OF WATER

Systems of Supply—In the preceding pages an attempt has been made to point out the solution of the problems in connection with the selection of a suitable source, system of purification and method of collection of water required for a community. It is now intended to discuss the various means of conveyance of water from the source or collecting basin, as the case may be, to the centre of the supply. For this purpose, engineers generally classify public water supplies into the following systems:—

(*i*) Gravitation System.
(*ii*) Pumping with intermediate storage system.
(*iii*) Pumping with direct pressure system.

Gravitation System—When an abundant and wholesome water supply can be obtained at sufficient elevation and within a reasonable distance from the town to be supplied, the gravitation system is certainly superior to all other systems inspite of the heavy outlay in works of collection. This system of supply, when well designed, is secure against all accidents, and maintains an uniform quality, quantity and pressure with the maximum possible safety. In this system, the water is collected in a natural or artificial reservoirs from a gathering ground, and then conveyed to the centre of supply by means of conduits.

Conduits, constructed for the conveyance of water for domestic purposes, should be impervious and covered, so that the water can be preserved from all contaminating influences. The selection of the type of channel or structure best suited for a particular situation depends upon several considerations such as safety, economy, topography, accessibility, material available, transportation, and opportunity for pollution. These may be made of brickwork, or stonework, plain or re-inforced concrete,

stoneware, steel, or cast iron pipes. Syphons are generally constructed of lines of cast iron, or steel pipes, and in crossing rivers or steam, the pipes are frequently carried on existing bridges or bridges specially built for the purpose. The form of the masonry conduits are decided by considerations of economy, facility of construction, and the load to be resisted, in addition to that of the hydraulic properties of the section. Fig. 68 shows some of the forms of the conduits actually constructed.

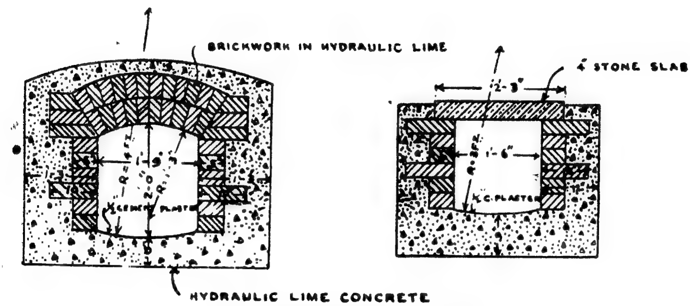


Fig. 68—Jharria Conduit.

The location of a conduit and the alignment of the route require careful survey and contouring of the locality made with this special object. A conduit laid along the contour of the natural ground is no doubt the cheapest. But, generally, this is not possible. Usually the hydraulic gradient is followed, wherever practicable, by contouring the sides and tunnelling the ridges or high ground, and only crossing the valleys or streams by pipes along the depressions as has been done in case of Jharria Water Works.

Where service reservoirs of sufficient capacity can be constructed at proper elevation, conduits are designed to deliver at least the maximum daily supply in 24 hours. They should be designed to give the maximum delivery when flowing half or two-thirds full. From the point of an hydraulic engineer, a circular form is the best shape of cross section of a conduit, as it gives the maximum hydraulic radius for a minimum area. A circular section, however, cannot always be used owing to

difficulties in construction, and in some cases, it is not adopted being not so suitable to resist loads which come on a conduit.

Pumping Systems—The genuine direct pressure system, generally speaking, has now passed into history, for no one in the present day conversant with the difficulties of waterworks management thinks of putting up a plant without intermediate storage to balance the sudden excessive demand, while the pumps are being accelerated to give the increased delivery. In this system, the sizes of the pumps need to be excessively large in order to meet the working condition, and the plant has consequently less efficiency for the greater period of working.

It is now almost the universal practice that every waterworks must have some sort of storage in the form of an elevated reservoir, or a stand pipe with reservoir to equalise the head against which the pumps are required to work ; thus their working becomes more efficient and economical. The life of the plant is also thereby considerably increased.

In the plains of Bengal, where water is available at much lower level than is required to be delivered to the consumers, pumping system is generally adopted.

A waterworks pumping station should as far as possible be close to the distribution reservoir to make the length of the rising main the minimum possible, and the reservoir should be centrally situated with regard to the distribution system to secure an even pressure throughout the system. The pumping station should be well above the highest flood level of the locality, and in a position free from dangers of fire from neighbouring structures. The building should be well drained and present generous accomodation for the comfort of the operators, and also for the safety of the equipment. The ventilation and lighting of all parts of the building should be perfect, so as to assure that no equipment shall be neglected, to serve as an incentive to careful operation, and to create a pride on the part of the operators in the appearance and maintenance of the plant. Equipment in a small, dark, unpleasant place is more easily neglected than equipments, which are easily accessible for observation, operation, and repairs.

Pumping station in this country may be classified either according (*i*) to the character of services rendered, or (*ii*) to the character of load to be handled. According to the former classification, a pumping unit may be either for a raw water, clear water, or both, or for a ground water supply. According to the latter, the units may be divided as follows:—

(1) Pumping at an uniform rate through a service reservoir of sufficient capacity to take up the daily variation. The pumps are large enough to meet the seasonal variation. In such stations, the peak load on the pump is about 25% above the daily average load.

(2) Pumping at a moderately varying rate through a balancing tank of limited capacity. The pumps provided are large enough to cope with the variation of consumption during a single day. In such cases, the peak load is usually 50% to 60% higher than the daily average.

(3) Pumping at a constantly varying rate without any reservoir. The peak load, in such cases, is 2 to 3 times as great as the average.

Of these, the first two types are generally found in practice and the last is gradually becoming rare.

The equipment in a pumping station consists of two or more prime-movers, which may be steam or internal combustion engines, or electric motors, and two or more pumps, and a full set of recording instruments, for such as, rate of pumping, delivery head, total lift, coal weighing, feed water measuring, steam pressure, flue gas temperature, feed water temperature, electric current consumption, power, voltage and also a small workshop, with lathe, drill and other necessary tools &c.

It is of paramount importance that the pumping machinery should be designed and arranged in such a way that it can perform its duty readily and in an efficient manner, and no expense should be spared in providing duplicate and standby equipment ready for immediate service. In all cases, pumping plant must be capable of coping with the maximum demand, unless the storage capacity provided is such, that an uniform rate of pumping can be maintained and any demand above it

can be supplied by the storage reservoir. If the pumps are made of sufficient size to meet the maximum peak load without any storage reservoir, as in the case of class 3, the average rate of pumping will be very low, and consequently, the plant will be working under most uneconomical condition for the greater part of the pumping period.

The efficiency of pumping stations depends on the suitability of size, and type of prime-movers, pumps, and other equipments, and their dispositions. The factors affecting choice of any type of plant include reliability, availability, initial cost and cost of maintenance and running. The best method of selecting a pumping plant is to consult firms of pump manufacturers, giving full details of requirements and conditions of working. These firms generally lay out large capital for perfection of their machinery, and take advantage of every improvement that can be obtained, either from scientific experts or money. Water-works engineers generally now-a-days content themselves with specifying the general outlines of the schemes and the results desired. But, at the same time, it is highly desirable that they should be able to decide generally which type of equipment will be suitable, so that they can direct their enquiries to those manufacturers who specialize in the types concerned. An expert in steam prime-mover is not likely to be an expert in oil or gas prime-movers, and the expert in centrifugal pumps is unlikely to be expert in reciprocating pumps and so on.

Energy required for pumping—The first question the engineer has to settle is, what prime-mover to adopt—steam, gas, oil, water-power or air ; or whether to use electric energy from a public electric supply corporation. This latter point will be taken up first.

Electric Supply—With the development during recent years of central station power plants generating energy at high efficiencies, and the increase in reliability of this power due to the interconnection of the various transmission systems, the advantages of using purchased power for small pumping stations within a reasonable distance from the central station are becoming more significant. These advantages are now being

generally recognised. Electric power can be purchased more cheaply than it can be generated in the largest pumping station ; consequently, generation of electricity in most of the waterworks in this country is not economical from the financial standpoint. Besides, the character of load furnished generally by waterworks is such that arrangements can be made to secure the most advantageous power rates. This can be done, because the peak load in waterworks generally comes at a time when the load on the electric power plant is low. For these reasons, operation with purchased power means minimum investment of capital, a minimum labour cost and relatively high operating efficiencies for units. With all these advantages, however, experience in Bengal shows that an offer of supply of electrical energy at half anna or less per unit is generally acceptible and anything above it requires careful investigation.

Electric energy is distributed either as a direct or alternating current. A direct current is the form of electricity in which the direction of flow along the conductor remains unchanged, while an alternating current signifies that the current alternates, *i.e.*, reverses or changes in direction along the conductor. An alternating current is said to have frequency of so many cycles a second, and in each cycle there are two alternations or reversals. The number of phase of an alternating current is represented by the number of different pulsations of current in one cycle. Most of the electric supply stations in this country deliver three phase alternating current of 50 cycles ; for retail consumptions, the voltage is generally 400, but higher voltages such as 3,000 to 6,600 are sometimes available without transformation. Direct current is generally available in 440 and 220 volts.

When direct current is to be used, the voltage of supply must be the same as that for which the electric equipment is designed, and the conveying capacity of the transmission line must be sufficient to enable enough current to flow through it to operate the equipment connected to it. In the case of alternating current, it is necessary that the equipments should be designed to work with the same frequency and phase as

the current on the transmission line. With regard to voltage, it may be either the same, as in transmission line, or may be reduced by putting in a transformer. The transmission line must be of sufficient capacity to convey the current required for the equipment to be connected. The cost of transmission of alternating currents is less than that of direct current.

In the selection of electric motors, the waterworks engineer must take into consideration the suitability of the types of motors for the condition of working, rating of motors, permissible over-loading under conditions of working, and other similar factors.

The types of motors generally used in waterworks are shunt wound direct current motors ; and induction and synchronous alternating current motors. Of these, shunt motors have practically constant field strength, and the speed variation from full load to no load is very small. A wide range of speed variation of 6 to 1 is possible by rheostatic control in case of direct current motors fitted with interpoles. This type of motor is suitable for constant working of load in a waterworks pumping station. Two types of alternating current motors are generally used in waterworks, *viz.*, synchronous and asynchronous. The asynchronous or induction motor, as it is usually termed, is the most popular type of alternating current motor. It has a very wide field of application. Induction motors are of two kinds, squirrel-cage and slip ring. The squirrel-cage motor is most frequently used because it is simple in design, rugged in construction and not easily damaged through negligent operation. It is also comparatively cheap to manufacture.

Slip ring motors do not differ a great deal from squirrel-cage motor ; the main difference is that the ends of the rotor winding are brought out to slip rings for connections, a feature which enables an external resistance to be connected to the rotor circuit to limit the induced current when starting. At no load, this machine runs practically at synchronous speed, but with the whole resistance out of circuit at full load, the speed is practically the same as that of the squirrel-cage motor. A

considerable range of speed can be obtained by varying the amount of resistance in the rotor circuit through a rheostat.

Ordinary synchronous motors operate at a constant speed and have the advantage of improved power factor. These motors are not self-starting, and they must be run to synchronous speed before the load is applied to the rotor shaft.

All alternating current motors being without commutators suffer from the disadvantage that they are fundamentally single speed machines, because the magnetic field rotates at synchronous speed. Sometimes, it is necessary to use varying speed motors driving varying speed pumps. In the past, the slip ring incuction motor with secondary resistance control through a limited number of steps resulting in considerable loss of energy has been the only alternating current motor available. The demand for a motor having speed regulation over a wide range with small increments and good efficiency over this speed range led to the development of the adjustable varying speed, brush shifting, single phase and polyphase motors—the so called A.C. commutator motors. Alternating current motors are now available to work safely with variation of speed in the ratio of 6 : 1.

The table below gives the particulars of efficiencies, etc. of different motors in practice :—

TABLE 38.

Horse Power.	Direct Current.		Polyphase Induction.	
	Speed R.P.M.	Efficiency at full load.	Power factor.	Efficiency.
3	900	82	80	80
5	800	85	82	82
7·5	650	87	84	84
10	600	88	85	85
15	550	89	86	85·5
20	500	89	87	86
50	400	90	89	88
100	300	91	91	89
300	200	93	92	90·5

Steam—Up to the beginning of the present century steam operated pumping plants predominated, and even now steam furnishes the motive power in the majority of installations in the world. Inspite of its requiring greater space for accomodation, more plant and accessories, and a largest number of operating staff, it has no competitor, where the plant is large enough to justify installation of steam turbines or multiple expansion engines.

Boilers—The first point an engineer has to decide in this connection is the selection of boilers. While doing so, the necessary power, the quality of feed water, the pressure to be maintained, the available floor space, and the quality and character of fuel are all to be carefully considered. The choice of boilers for waterworks in this country is not so wide owing to the size of installation not being great in comparison to those in other countries. The choice mainly rests with the shell type or water tube type of boilers. In the former classes are included Cornish and Lancashire boilers. These boilers are generally used for comparatively small installations ; they have large thermal storage and are thus useful for plants where sudden demands of steam have to be met ; although in waterworks practice, no serious difficulties of this nature is usually encountered. Their chief disadvantage is the small size of combustion chamber, which results in the volatile gases coming in contact with the water cooled surfaces and preventing the completion of combustion. Their internal dimensions make them easily accessible, and permits the use of water, which could not be used with types having more restricted space. Cornish type boilers vary in sizes from 4 ft. to 6 ft. 6 inches in diameter and in length about 10 ft. to 24 ft. Such boilers can evaporate up to about 3,000 lbs. of water per hour and work up to 150 lbs. pressure per sq. in., but 80 lbs. per sq. in. in more common. While Lancashire type of boilers are made in sizes from 20 ft. long by 6 ft. diameter to 30 ft. long by 9 ft. 6 in. diameter, with an evaporation ranging from 3,000 lbs. to 9,000 lbs. of water per hour, Lancashire boilers can be constructed for pressure up to 200 lbs. per sq. in. ; but it is not usual for the working

pressure to exceed 160 lbs. per sq. in. The thermal efficiency of a Cornish boiler with hand firing, natural draught, and economiser is about 60% and without economiser is about 45% to 55%, while that of a Lancashire boiler working with economiser, superheater, natural draught, and hand firing is about 70%, and that of large Lancashire boilers with mechanical stokers instead of hand firing varies from 68% to 78%.

In waterworks practice, the facility of transport, erection, smaller space, and the quickness of putting a boiler into operation are generally factors which in many cases determine the selection of a water tube boiler in preference to the shell type boilers. Water tube boilers can be constructed for any pressure up to 1,500 lbs. per sq. in. with an evaporative capacity of 200,000 lbs. of water per hour. They can be fitted with grates of considerable size, and can be provided with any desired ratio of heating surface. Further, their small water capacity enables steam to be raised to full pressure, if necessary, within a very short time.

The thermal capacity of this type of boiler working with mechanical stoker, air heater, balanced draught, and powdered coal is about 88%.

The efficient working of boilers of all types demands handling of fuel and ashes at minimum expense, burning the fuel as completely as possible with a minimum quantity of air, and transferring heat of the gases of combustion to the water with minimum loss during transmission. To attain these objects, boilers are required to be equipped with large furnace capacity, mechanical coal conveyers, mechanical stoker, air heater, forced draught, fuel economiser, and super-heater. Besides these accessories, when the feed water contains such impurities as magnesium sulphate, which cause the formation of scale on heating surfaces, water-softening apparatus may be necessary. Continued existence of the clean metal walls of heating surface is essential for efficient transmission of heat from the fire to the steam. Formation of scale on heating surfaces reduces transmission of heat and lowers the boiler efficiency. When

condensing reciprocating engines having cylinder lubrication are used, efficient oil separator is also to be provided so that oil is kept off from the condenser and from the boiler feed water. Deposition of oil on a heating surface is dangerous.

Regarding economy of fuel, hand firing by a skilful fireman is often the best and cheapest, especially so when boilers are worked singly. Where however groups of boilers are worked and fuel is distributed to the hopper by mechanical conveyers, a considerable saving of labour may be effected by mechanical stoking. On the other hand, mechanical appliances require more skilled attention and the cost of wear and tear is heavy. With mechanical stokers, a lower grade of fuel can be used which is not otherwise possible, and also the fact that fuel can be supplied into the furnace without opening the door is a further advantage. There are several types of stokers in the market, and all seem to be equally efficient for the need of steam users.

For perfect combustion, a pound of coal requires a definite amount of air. The amount of air, which will pass through the bed of the fuel, depends on the air pressure available to force it through. The suction in the chimney varies directly with the height of the chimney as well as the temperature of the gases within the chimney. The maximum suction, which a chimney of given height can produce, is fixed ; when this height of the chimney cannot, for certain reasons, be arranged, it is necessary to suppliment the chimney draught by mechanically induced or forced draught (Fig. 69). In large installations, it may be useful to provide for induced forced draught fans which enables inferior grade of fuels to be burnt. In Indian water-works practice, however, natural draught of a chimney is generally found to be sufficient, but it is all the more important that the chimney should perform its function adequately under all conditions of weather and load. Chimneys cannot be designed with precision, because of the uncertainty of the various factors involved. Many empirical formulae are in use, and the discrepancies between their results are wide. The height and design of a suitable chimney may generally be left to the supplier

of the pumping machinery, as their experience under similar working conditions is often valuable. When expert advice is not available, the following thumb rule may be useful. The area of the chimney may be made equal to one tenth of the grate area or equal to an allowance one seventieth of a square foot per pound of coal burnt. The height is generally fixed by local bye-laws and seldom less than 70 feet.

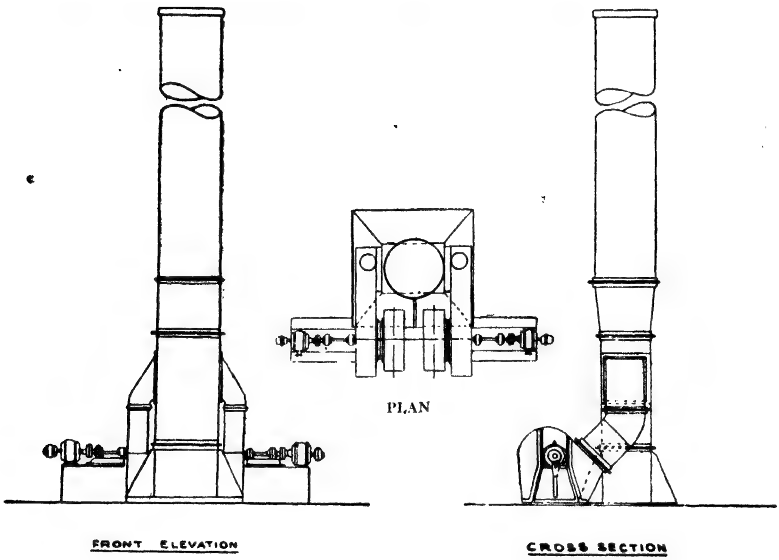


Fig. 69—Steel Chimney with induced draught fan.

The height and area can also be found by one of the following formulae based on experience.

when, A = area of the chimney in Sq. foot.

h = height of the chimney in feet.

F = total pounds of coal burnt per hour.

According to Smith,

$$A = \frac{0{\cdot}0825\ F}{\sqrt{h}}$$

According to Kent,

$$A = \frac{0{\cdot}06\ F}{\sqrt{h}}$$

Economisers—In the generation of steam in boilers, the escape of high temperature flue gases by the way of chimney is a source of considerable waste. This waste may be minimised by the installation of a fuel economiser, the function of which is to recover heat from the waste gases and impart it to the feed water before it enters the boiler. By its aid a saving of 10% to 15% of the fuel can be effected. A waterworks installation of reasonably large size in this country is generally fitted with a Green's economiser (Fig. 70). This economiser consists of a

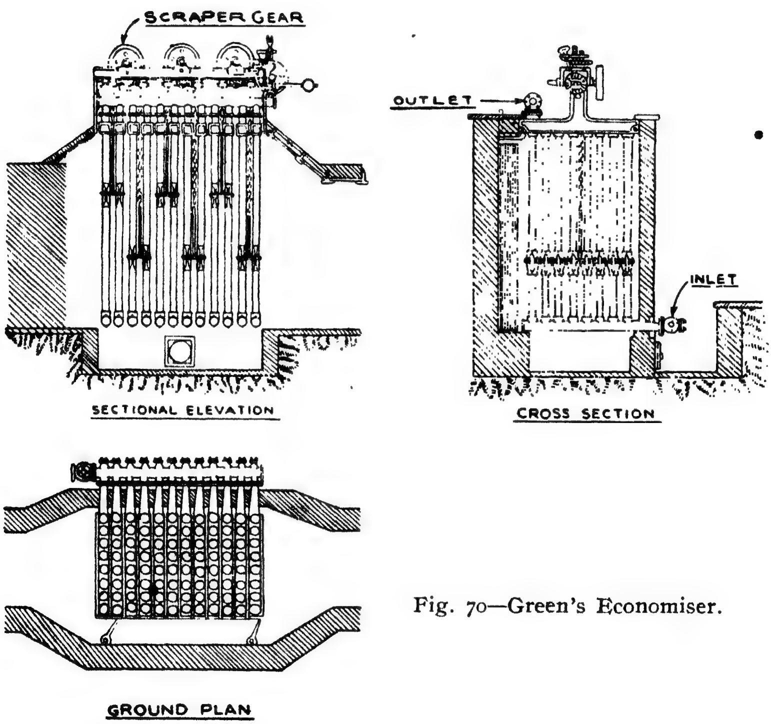


Fig. 70—Green's Economiser.

stack of vertical cast iron tubes $4\frac{9}{16}$ ins. external diameter and 9 ft. to 13 ft. in length and connected together in rows by boxes at the top and bottom. The various rows, being coupled together by longitudinal rows of pipes at the diagonally opposite sides, form respectively the feed and delivery pipes of

the apparatus. The feed is introduced at the end of the apparatus nearest the chimney, so that the flow of water and gases are in the opposite direction. By its aid the temperature of the waste gases is reduced by about 300 deg. Farh. and that of the feed water is increased by 150 deg. Farh. The life of this economiser is about 15 years.

There are other forms of feed water heater, which are useful where it is undesirable for economic or other reasons to install an economiser. In these, generally, the temperature of the exhaust steam is utilised to raise the temperature of the feed water. They usually consist of a vertical cylinder, a series of tubes, through which either exhaust steam or feed water is passed. In the former type, the feed water circulates around and between the tubes and is admitted at the bottom and taken at the top. In the other type, the exhaust steam takes the place of feed water and feed water the place of steam. These types of feed water heaters are not suitable for water-works pumping stations, which are operated with the use of condensers delivering exhaust steam at a low temperature. Roughly speaking, for every 10 deg. Farh. that feed water is heated, there is a saving of 1% of fuel.

Superheaters—The advantages of using superheated steam is now recognised. A superheater is essentially a boiler, in which live steam is heated in place of feed water. The principal object of a superheater is to raise steam to higher temperature than its saturation point without increasing its pressure. This reduces the loss due to the cylinder condensation, and condensation in steam piping, and thereby increases the limits of the working temperature of the engine.

From 10% to 20% of economy can be obtained with reciprocating engines using a moderate amount of superheat say 150 deg. above the saturation temperature while superheated steam increases the efficiency of steam turbines by 8% to 10% per 100 deg. Farh.

The design of a superheater depends on the type of boiler with which it is used. Superheaters for use with boilers of the shell type like Lancashire and Cornish boilers are generally

in the form shewn in the Fig. 71. The superheater that is usually used with water tube boilers is shewn in Fig. 72. A

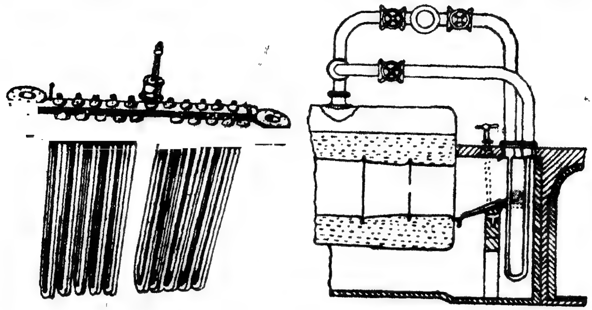


Fig. 71—Superheater.

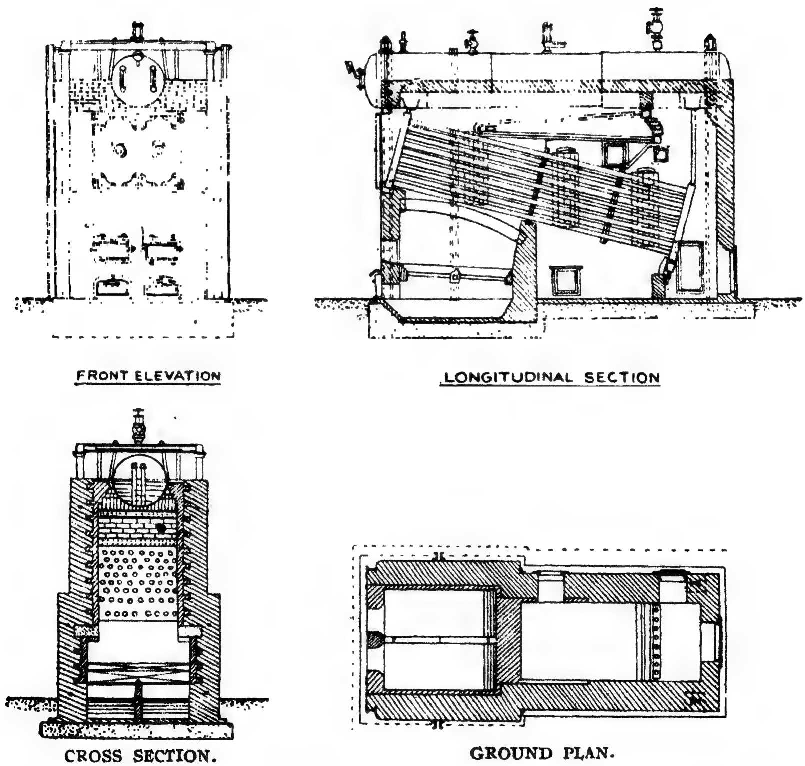


Fig. 72—Babcock and Wilcox boiler with superheater.

superheater, generally speaking, consists of series of bent coils of steel tubes which receive the steam from boiler stop valve

on the steam drum, and lying in the path of gases before they are cooled, adds temperature to the steam before delivering it to the steam pipe, keeping it dry, increasing its volume and enabling it to be expanded to a greater degree before condensation occurs.

The waterworks engineer should ascertain that adequate arrangements have been made for flooding the superheater when required, that isolation of superheater is simple, that the thermometer pocket is accessible, that the temperature can be read without discomfort, and that the replacement of burnt tubes is not impossibly difficult.

Fuels—The fuels generally used for steam generation in this country are coal, coal dust; and liquid fuel, of which coal is most commonly used in Bengal. The use of gaseous fuel and wood is so unusual in waterworks practice as to be of any general interest.

Coal is the most valuable fuel in industry ; its availability, and quality to a large measure determine the location of industries and also to no inconsiderable extent the prosperity of a town. Coal is selected on the basis of its heating value, the amount of ash and its burning characteristics. The Government of India has appointed a Coal Grading Board, who issue a classified list of all coals available in the Barakar and Raniganj coal fields graded by their calorific values and other leading characteristics. Graded coals are primarily classified as *low volatile coal* and *high volatile coal.* Each of these classes are divided into four grades, (i) Selected grade, (ii) Grade I, (iii) Grade II, and (iv) Grade III, according to the characteristics, given in the following table.

TABLE 39.

Grade.	Minimum calorific value.	Maximum per cent. of ash.	Maximum per cent. of moisture.
Selected.	7000	11	6
Grade I .	6500	13	9
,, II .	6000	18	10
,, III .	Any coal below the above standard.		

The heating value of Indian coal varies from 13000 to 5000 B.Th.U. per lb.

The use of pulverised fuel is coming more and more to the attention of the steam power producer, and the time may not be far distant when it may be generally adopted in water-works pumping stations. To use pulverised fuel, the raw coal is first crushed into lumps of about 1 inch cube and then dried to reduce the moisture contents to 1% to 3%. After drying, the fuel is passed into a pulverising mill and reduced to fine powder, so that about 95% of it passes through 100 mesh screen and 75% through 200 mesh screen. The use of pulverised fuel has the advantage of considerable flexibility of operation (nearly approaching that of oil fuel) owing to the easy control of fuel supply to the furnace, no banking or standby losses. Further, the air supply can be controlled, and minute particles of fuel can be directly fired with the requisite quantity of air. This advantage makes it usually possible to obtain a boiler efficiency of 90%. Volatile fuels of high heat value and low in ash are the most desirable as pulverised fuel. But nearly anything can be burnt, even low grade lignite.

The use of pulverised fuel, however, involves the trouble and cost of an independent pulverising mill, and also the objection that the ash in the fired fuel is carried forward by the draught and sometimes expelled from the chimney top creating nuisance in the neighbourhood.

Liquid fuels possess superior heating value, weight for weight over coal of from 30% to 60%. It is also readily stored and its use involves the employment of only one quarter of the attendants required in coal-fired plants. The heating value of coal depriciates with the period of storage and exposure while oil maintains its calorific value indefinitely. An oil fired furnace can be easily raised to its maximum temperature from cold, and easily maintained at its full heating capacity for prolonged periods without variation. Firing can be instantaneously stopped, standby losses being accordingly avoided.

The perfect combustion readily obtainable with fuel oil means absence of smoke, soot, or ashes, while the facility, with

which it can be pumped into storage tanks, secures absence of dirt troubles and labour associated with handling of coal. For efficient combustion, oil fuels require large combustion chamber and suitable apparatus for thorough atomisation.

Three types of burners are usually used in this connection for atomisation, (i) steam jet burner ; (ii) air pressure jet burner ; (iii) mechanical atomising burner. In steam jet burners, the oil is vaporised by being mixed with a jet of steam close to the burner nozzle, through which the fuel is delivered into the furnace immediately before combustion. With air jet system, the compressed air from a receiver is used as the atomising agent. The oil is fed to the burner by gravity from an overhead tank containing heated oil and is brought into contact with the compressed air heated by boiler flue gases or by other means, and is thereby completely vaporised and injected in the furnace.

In mechanical atomising burners, the oil is vaporised by being broken into fine spray and thrown out in the form of a hollow cone by the working of an impeller inside the atomiser.

Of these, the steam atomiser is simpler in design, cheap in first cost and in the cost of working, and high efficiencies can be obtained at low and moderate ratings. It has, however, the disadvantage of relatively high steam consumption at the burner ; the boiler capacity is reduced thereby.

In selecting oil for fuel purposes, in addition to the calorific value of the oil, its viscosity, its minimum water and sulphur contents and sediments should be taken into considera-tion, so that the fuel may not require undue heating for pumping and atomising. The following physical characteristics of the fuel must also be considered,—specific gravity, flash point, specific heat, and chemical composition. The flash point of a fuel must not be below 150 deg. Farh., otherwise there is danger of conflagration from storage.

Under favourable conditions, 1 lb. of oil, generally speaking, will evaporate 14 lbs. to 16 lbs. of water from and at 212 deg. Farh. Whilst, under similar circumstances, a pound of best coal will evaporate 7 lbs. to 10 lbs. of water.

Steam Pumping Plants—Steam driven pumping engines may be divided broadly into two classes:—

A. *Rotative class which comprises*:—

(a) Beam-engine with or without fly wheel.
(b) Crank and fly wheel engines.

B. *Direct acting engine which may be of*:—

(a) Simple form.
(b) Differential or Compensated type.

The last three types of engines may be horizontal or vertical. Beam-engines were once considered to be the most efficient type of waterworks plant, and a large number of them was installed in works constructed between 1836 to 1885. This type of engine has gone out of fashion, and is no longer manufactured. In a few cases, however, the beam is still retained and a heavy fly wheel is added. The crank fly wheel type engines are not suitable for standardisation, and must be designed to suit individual cases. A typical modern engine of this type would require 12 lbs. to 15 lbs. of steam per B.H.P. hour.

Direct Acting Engines—Simple (or non-compound) direct acting non-condensing steam plants generally use steam too extravagantly to be of much use in waterworks practice. Compound forms have been used in small plants, but they are also not considered very economical. Direct acting condensing engines now generally used are of triple expansion type, arranged either horizontal or vertical. The distribution of steam, however, requires special arrangements, as while the pressure in different cylinder is varying, the pressure in the water cylinder is constant. There are numerous devices of this kind in the market; one of them is found in Worthington Simpson pumping engines with oscillating compensating cylinders, which store power during the initial pressure portion of the stroke and let it out during expansion portion of the stroke. The vertical triple expansion pumping engine (Fig. 73) is noted for its extremely high economy, reliability, durability, and flexibility, which allow satisfactory operation over a wide range of capacity and head without serious drop of economy.

The unit consists of a massive slow speed triple expansion engine set vertically over three pumps supported, either partially or wholly, by the chamber composing the pump. A crank shaft and fly wheel are introduced with connecting rods to crossheads on the steam side to equate and control the motion of the whole system.

In the horizontal (Fig. 74) direct driven types, it is quite normal to dispense with the crankshaft and the flywheel putting the cylinders in tandem and controlling the motion by an air cushioning piston and cylinder oscillating about a central pivot tending to retard motion at the commencement of a stroke and accelerate it at the end ; this enables the decreasing pressure in the steam clyinders to balance the constant pressure in the water cylinders.

Yet a third type combines horizontal tandem steam (Fig. 75) cylinders with vertical pump cylinders, motion being transmitted through a bell crank or rocking triangular lever device. This type is usually provided with crank and flywheel, and while not such a good mechanical arrangement as the vertical, has the advantage of leaving a deep pump, well accessible and open to the air, often a matter of great importance.

The mechanical efficiency of a triple expansion engine varies from 78% to 84% at full load ; the higher efficiencies have been generally obtained in vertical engines. The steam consumption in these engies vary from 10 lbs. to 15 lbs. per per pump horse power hour. A very much cheaper type of steam pumping plant is the steam turbine driving a centrifugal pump, either by direct coupling or by double helical gearing.

It is not usual to intall steam turbines for other purposes than pumping in sizes less than 750 or 1000 H.P. But for pumping, it is worth considering for much smaller sizes, say down to 150 H.P. owing to the small capital cost and small space occupied by it. Every type of steam turbine is applicable to pumping work, but the cheaper one or two-runner impulse types, while wasteful in steam, realise to the full the economy in capital cost which is the object of the installation. A consumption of 18 lbs. of steam per P.H.P. hour can be attained

with quite a cheap and small type, and 14 lbs. or less with larger types having more stages.

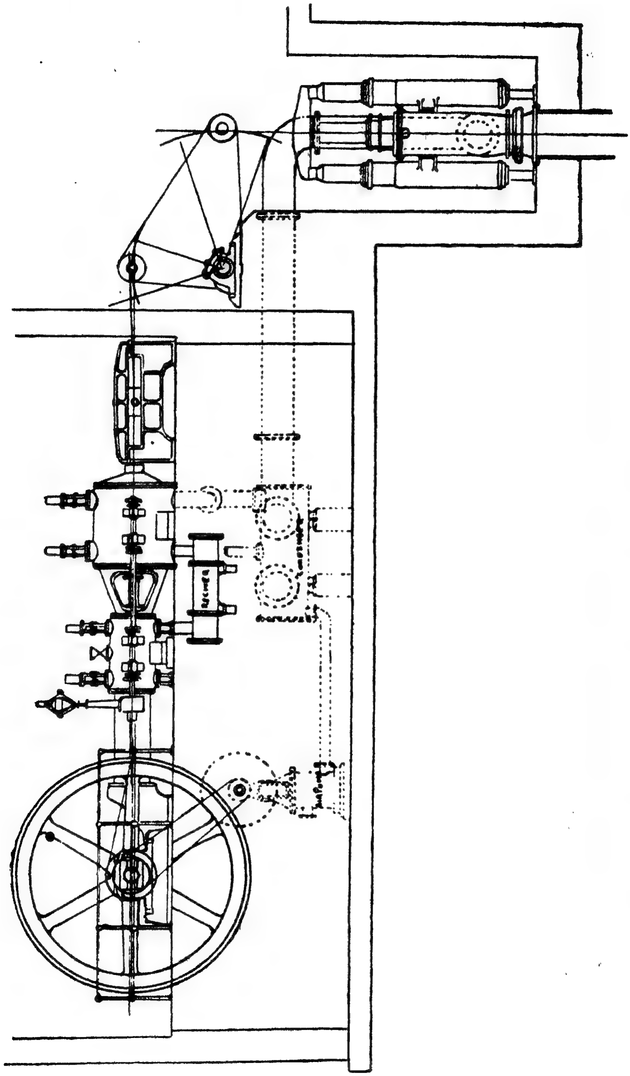


Fig. 75—Horizontal Tandem Engine driving Vertical Pump.

Great attention should be paid to accurate alignment, forced lubrication and circulation of oil, water cooling of bearings, etc. when installing a steam turbine and centrifugal

or axial flow pump. Once correctly installed, there is very little to attend to, and it is one of the most reliable units possible.

The steam turbine (Fig. 76) has a great advantage in high pressure work, the pressure on the pipe joints is continuous and not fluctuating, a desideratum never fully attained by any reciprocating pump, however well it may be served by air vessels. In practice, this results in tighter joints, less loss by leakage and longer life to the pipe line. When, it is very long, this is of great importance.

Condensers—In waterworks pumping plant layout, the ability to obtain excellent cooling by the use of the water pumped is generally taken advantage of by provision of a surface condenser large enough to pass the whole supply without appreciable loss by friction head.

The usual form is a long cylindrical box containing two disc diaphragms, one near each end, into which a large number of tubes are expanded, sufficient to pass the whole supply from one end to the other. Exhaust steam is admitted to the space between the diaphragms and outside the tubes where it condenses by contact with them.

A vacuum pump or ejector to exhaust this space is provided, while from the bottom of it a connection runs to a small condensate extraction pump, whose function is to remove the steam condensed as water and return it to the boiler hot well. Sometimes, the function of the vacuum pump and condensate extraction pump are combined in one air pump; this is specially the case in reciprocating plants where this plant is driven by an extension of the piston rod, or a crank on the main shaft. If a steam ejector is used, the heat of the steam is recovered by condensation in a similar smaller condenser cooled by water on its way to the boiler feed pump. A very simple ejector can be worked by water from the delivery pipe in some instances, the effluent returning to the suction pump. The condensate extraction can also be worked this way.

Fuel Consumption—The following quantities of coal are usually required for different types of steam pumping engines ;

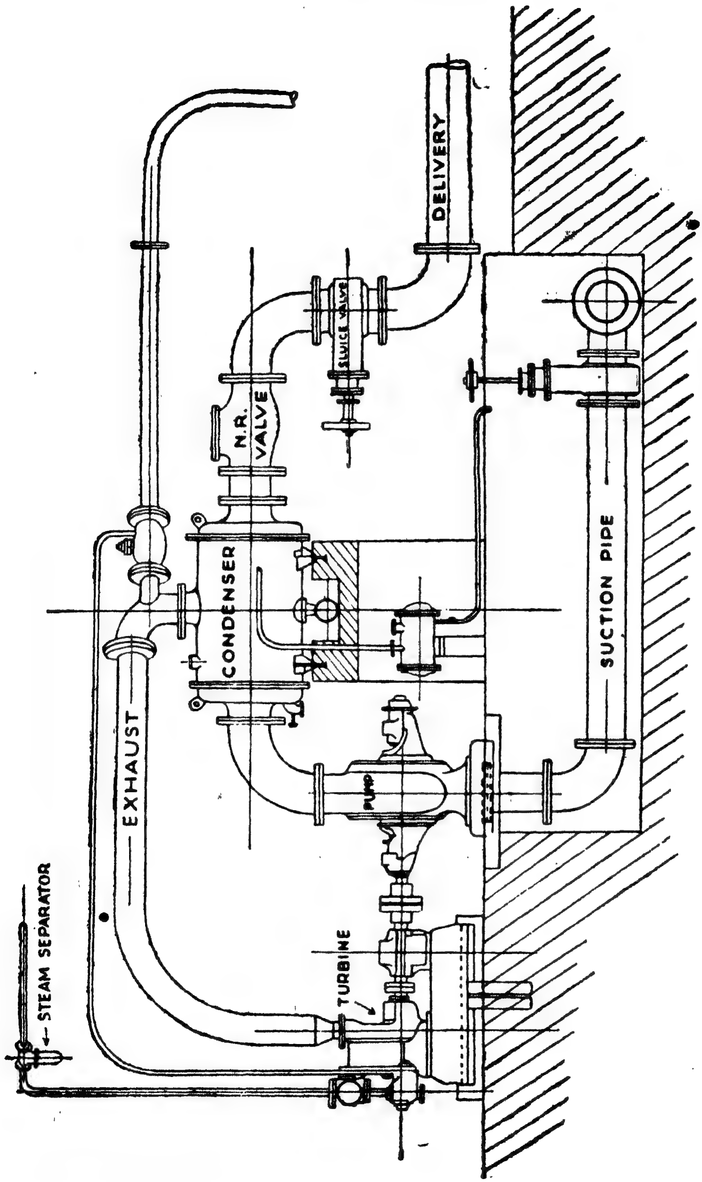


Fig. 76—Turbine driven Centrifugal Pump.

these figures are based on the assumption of evaporation of 6 lbs. of water per lb. of coal.

TABLE 40.

Types of Steam Pumping Engines.	Coal consumption per P.H.P. hour in lbs.
Non-condensing Engines	17 to 20
Condensing Engines	9 ,, 11
Compound Non-condensing Engines ..	7 ,, 9
Compound Condensing Engines	5 ,, 6
Triple Expansion Condensing Engines ..	2 ,, 4
Steam Turbine	3 ,, 4

Oil Engines—In more recent years, another type of primemover has come into general use for driving pumping units ; this is known as the oil engine. One of the principal reasons for using oil engines in pumping operations is that when all things such as, first cost, repairs, maintenance, etc. are taken into consideration, they are usually more economical than with other types of drives, within certain range of the sizes of pumping plants. It is interesting to compare the figures of average consumption of heat units of various types of prime-movers used for power generation in both industrial and central station plants. Mr. Baker in his paper on "The Diesel Engine as an economic factor in pumping" gives the following thermal results obtained in different primemovers.

TABLE 41.

Types of Engines.	B.Th.U. Consumed per Brake Horse Power.	
	Full Load.	Three Quarter Load.
Simple Non-condensing Engines ..	52,000	58,000
Compound Condensing Engines ..	23,000	28,000
Steam Turbines	20,000	23,000
Oil Engines	8,000	9,000

The use of oil engine depends to a very great extent on the supply cost, carriage and availability of suitable oil in comparison to coal. In India, oil suitable for use in an oil engine,

having a thermal value of 18,500 units per lb., can be had at almost all the principal ports at Rs. 50/- or Rs. 60/- a ton, while coal, of about 13,000 B.Th.U. per lb., can be had at the pit mouth at a cost of about Rs. 5/- per ton, or in other words, the cost of oil is 7 times more than that of coal for the same thermal units.

Taking everything into consideration, it is difficult to justify any oil engine plant for waterworks for sizes over 200 B.H.P. where good coal and efficient steam plant is procurable at a reasonable cost, but the extra capital cost of boilers and auxiliaries with high steam consumption militates against the use of steam plant in sizes under 160 B.H.P.

The following extracts from American Water Works Practice regarding the use of oil engines in waterworks will be found interesting :—

"The very low fuel consumption of the oil engine in combination with a reciprocating or centrifugal pump makes a very economical arrangement. The fuel consumption of the full Diesel Oil Engine will usually be around 0·48 pound of oil per brake horse-power-hour, while the Semi-Diesel will consume about 10% more, and the consumption does not vary greatly over a considerable range of speed or head. Oil engines are capable of satisfactory operation at one-half to full speed, but should not be expected to carry substantial overloads. The overall duty obtained in the Diesel engine driven pumping unit is materially higher than what can be obtained by any other type of pumping unit with the exception of large triple expansion pumping engines. This point is of importance, when considering the smaller sizes of pumping units. Even the small Diesel engine driving a reciprocating pumping unit will develop upwards of 200,000,000 foot pounds per million British Thermal Units." The fuel consumption of the most modern Diesel Engine varies from 0·38 to 0·41 lbs. per B.H.P. hour.

Of the many types of oil engines in the market, Diesel and Semi-Diesel engines are generally used in waterworks ; any of these can be either two or four cycle.

Diesel engines have got very high thermal efficiencies, and can be run with almost any kind of liquid fuel ; they are however expensive in first cost and are comparatively difficult to operate. They can be run under considerable variation of load and range of speed. They require about 0·38 to 0·415 lbs. of crude oil per B.H.P. hour. The thermal efficiency is also almost constant over a range of considerable load variation.

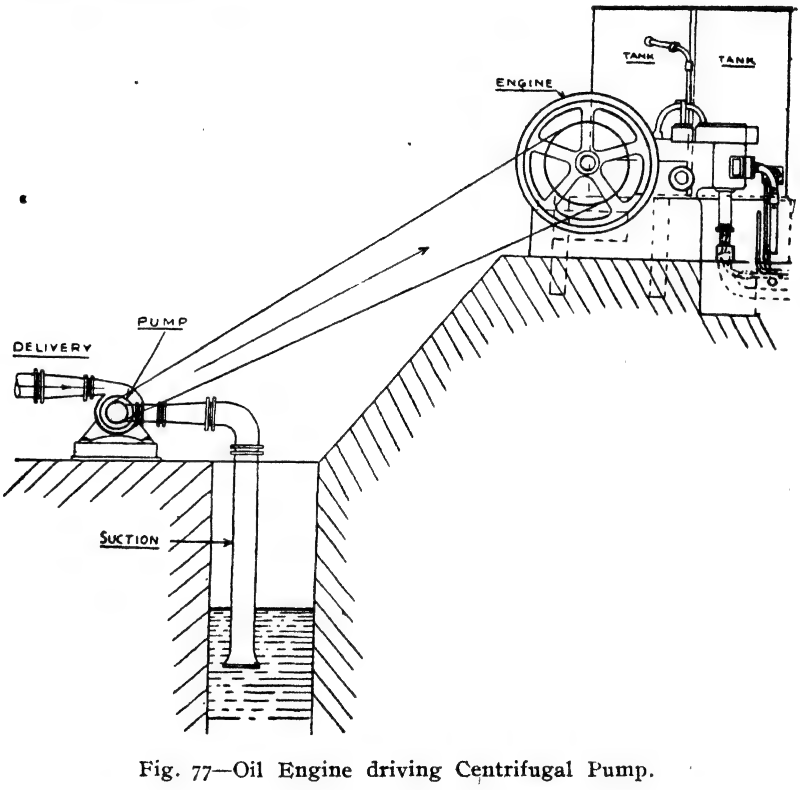


Fig. 77—Oil Engine driving Centrifugal Pump.

Semi-Diesel engines are lower in first cost, especially in smaller sizes ; the cost of operation of these engines is less than diesel engines, and have higher mechanical efficiency, but lower fuel efficiency. They usually work with about 0·45 lb. of crude oil per B.H.P. hour.

In comparing of the features of four and two cycle engines,

there is much to say in favour of the former, although the latter has got fewer valves to handle and requires less space and lighter foundation. The four cycle engine can be operated at higher speed with better economy. The four cycle engine is comparatively easy to operate and less liable to break-down owing to careless operation.

The speeds of oil engines range from 350 revolutions per minute in smaller sizes to 200 R.P.M. in larger sizes of engines. The mechanical efficiency of oil engines vary from 70 to 85 per cent. but is generally taken to be 66 per cent.

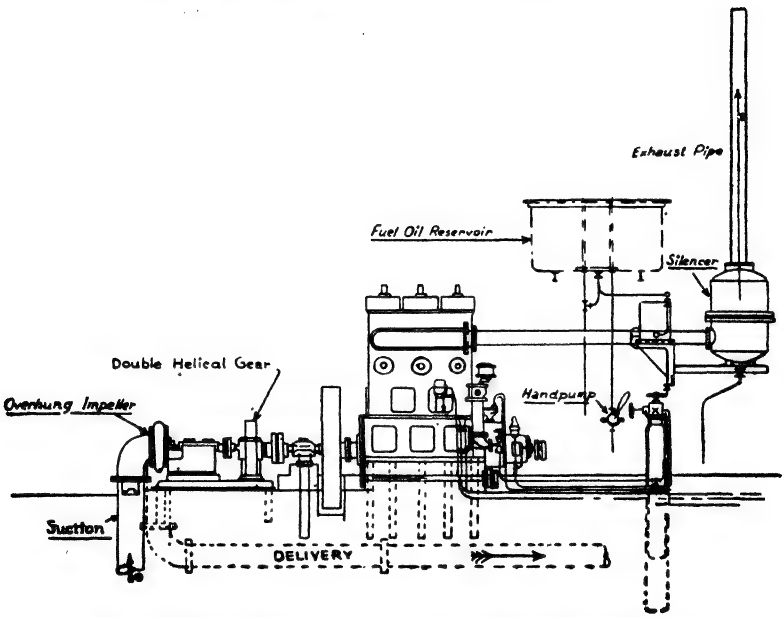


Fig. 78—Oil Engine driving Centrifugal Pump through helical gear.

In the majority of earlier installations using oil engines to drive pumping units, the engine was connected to the pump by means of a belt (Fig. 77) and even to-day this plan is followed in many cases. However, in recent years developments in oil engine pumping plant have departed from belt drive, and the engine has been connected to the pump, either through gears (Fig. 78) or other speed changing devices.

For example, in case of Duplex plunger pumps, the engine shaft is coupled to the main shaft which is in turn geared to the plunger crank shaft, thus permitting the economical engine speed without interfering with the required pump speed. Another type of oil engine driven pumping installation consists of a large triplex plunger pump with engine directly connected to the main shaft through a flexible coupling ; this arrangement is only possible when the speed of the engine corresponds to the speed required for the pump.

In case of a high speed centrifugal pump, which requires to be run at a speed higher than that of the engine, it is of course necessary to introduce speed changing devices that will permit the proper speed ratio between engine and pump. Devices, such as double helical gearing, are not only highly efficient, but are absolutely dependable, and when proper care is exercised during installation, no difficulty is experienced.

Pumps.

Reciprocating Pumps—The pumps in common use in waterworks practice are either reciprocating, rotary or air lift. The simplest form of reciprocating pump consists of a cylinder with inlet and outlet openings controlled by suitable valves and a piston having reciprocating motion working inside the barrel.

There are various types of this class of pump, and each type can be arranged to operate either horizontally or vertically and may be single or double acting.

Of these, bucket pumps are those, in which the water is drawn through a suction pipe during the ascending stroke of the piston and delivered through an outlet valve fixed in the piston. Piston pumps have solid pistons, and the inlet and outlet valves are fixed on the barrel.

Differential pumps are single acting on suction side, and deliver water in both strokes.

The delivery from single acting pumps is fluctuating, while that from the double acting pumps is more continuous.

Reciprocating pump may be driven by pulley and belting, chain drive, or gearing, and are generally called power pumps, while those operated by steam or other medium working on another piston (moving inside a second cylinder) coupled together by the one and the same piston rod of the pump are called direct acting. The range of speed of power pumps varies from 20 R.P.M. to 120 R.P.M., the slower the speed, the less the wear and tear, and the life is longer. The delivering capacity of those pumps varies with the diameter and length of the stroke of the piston and is independent of the height to which the water is raised. Normally, the length of the stroke is made equal to the bore of the cylinder, and in short stroke pumps, the stroke may be anything upto ¼th. of the bore, and in long stroke ones, it may be 2½ times the bore. The speed of the piston is also another important factor in the selection of the pump. In practice, the common speed generally adopted is about 100 ft. per minute, but a good speed of 250 to 350 ft. per minute is not very uncommon, and is more the modern practice.

The barrels, plungers and valve boxes are generally made of cast iron, lined with gun metal. In small size pumps the plungers are frequently made of solid bronze. The plunger rods are made of steel and the stuffing boxes, and plummer blocks supporting the shafts are either made of gun metal or cast iron lined with gunmetal. In good design, the valves are made easily accessible and liberal in size of opening. The moving parts of the pump should be light in weight, so that the power required to overcome the inertia in them is reduced to the minimum. The motion of these parts should as far as possible be slow. To reduce slip, the valves should close rapidly and fit accurately on their seatings, and be ample in size. The piston must fit the cylinder or plunger its ring as closely as practicable with minimum friction. The pump as a whole should be substantially built, and should be so designed that it is not affected by heat during working, and all its moving contact surfaces are efficiently and automatically lubricated.

Reciprocating pumps should not be placed more than 20 ft. above the lowest suction level, although sometimes they are fixed 24 ft. to 26 ft. above it. When the suction is more than the maximum, the pumps usually are put in a pit to bring it within the easy suction distance of water.

Reciprocating pumps are generally provided with the following accessories:—

(i) Footvalve, and strainer at the end of the suction pipe with priming arrangement for easy starting. (Fig. 79).

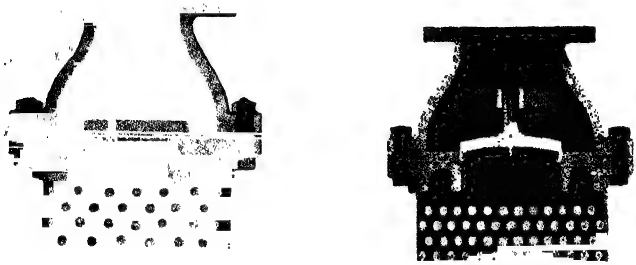


Fig. 79—Foot-valve and Strainer.

(ii) Suction air vessels are generally fixed in modern pumps with the object of making the velocity of flow in the suction pipe uniform, and thereby to reduce water hammer and valve shock to a minimum. This is a very important point.

(iii) Delivery air vessels are provided above the delivery valves to promote silent running of the pump by reducing vibration in the pump due to variable loading.

(iv) Relief valves are the safety valves of the system and save the pump from damage, in case the delivery sluice valve is closed when starting.

(v) Bye-pass valves are also provided for priming, relieving a pump of pressure when starting up, or for draining the different chambers of the pump.

(vi) Suction and delivery pipes are usually made about one-half the diameter of the barrel, the suction being made slightly larger than the delivery.

The following points should be considered in selecting this type of pump :—

(a) strength and simplicity of different working parts ;
(b) long stroke and ample wearing surfaces ;
(c) positive action of valves which should have ample water-ways with small lift ;
(d) a continuous flow of stream of water in the whole stroke ;
(e) capable of running with steadiness at all speeds ;
(f) capable of being started in any position of the piston ;
(g) parts to be simple in design and easy of access and adjustment ;
(h) capable of smooth running at all speeds without pounding ;
(i) low absorption of power.

When dealing with considerable pressure, the reciprocating pump, when well designed and working at a fairly slow speed, is capable of giving an efficiency of about 90%, but as the working head is reduced, the efficiency falls off rapidly so that for heads below 100 ft. centrifugal pumps become more efficient. The piston pump, however, is more positive in action than centrifugal pump, whose power of delivery depends largely on the periphery speed of the impeller.

Centrifugal Pumps—Centrifugal pumps occupy an extended field of usefulness, and it is now generally accepted to be the best type of pumps for waterworks service. They are now commercially available for any capacity and for heads over 1000 ft. They are also now being manufactured to work under variable conditions of head and delivery. In comparison with the reciprocating pump, this type of pumps for a given duty are less costly and easier to install ; they give a continuous flow of water without the trouble of air vessels. They occupy relatively small space, and are adopted to high speed primemovers. Having no valves or glands to go out of order, they are less costly

to maintain. They can pump almost any kind of liquid, which need not be free from grit or solid as in the case of the ram pumps.

In its simplest form a centrifugal pump consists of (i) a casing with suction and delivery openings ; (ii) the impeller which closely fits the casing and revolves within it ; (iii) the spindle or the shaft—the means of transmission of energy and motion to the impeller from the primemover. The function of the casing is to fully enclose the impeller, and to collect and effectually force out the water so collected from the periphery of the impeller through the delivery opening. The function of the impeller is to effectively transmit the energy derived from the spindle to the water to be pumped.

Centrifugal pumps are usually of two types (a) volute pumps (Fig. 80) ; (b) guide vane pumps (Fig. 81). In the former, the vanes of the impellers are spirally arranged with gradually increasing area to suit the volume of water delivered. In the latter, between the impeller and the collecting chamber diffusion vanes are introduced.

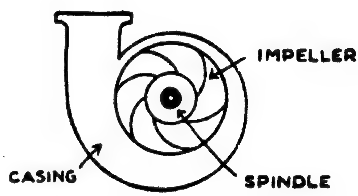


Fig. 80—Volute Pump.

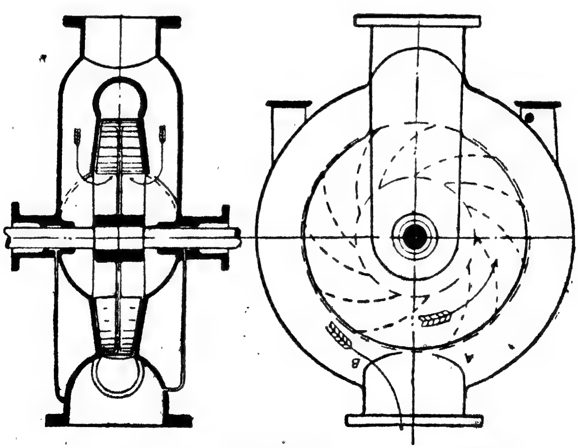


Fig. 81—Guide Vane Centrifugal Pump.

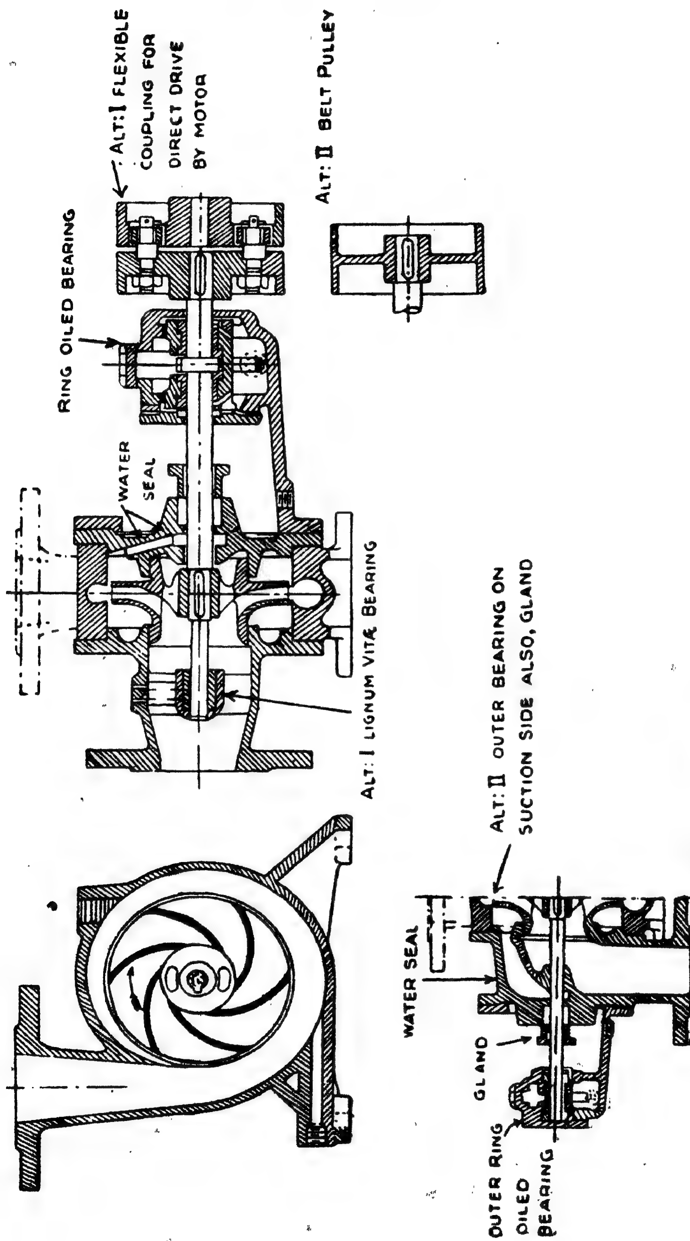


Fig. 82—Single stage low lift Centrifugal Pump.

Of these two types, volute pumps are generally constructed for low and moderate pressures, especially when a large quantity is to be pumped against small head, while guide vane pumps are intended for high pressure supplies, and where the volume to be delivered in comparison to head is small. Centrifugal pumps designed to raise water up to a head of 80 ft. are termed low lift (Fig. 82), up to 200 feet are called medium lift (Fig. 83) and above 200 ft. are called high lift pumps.

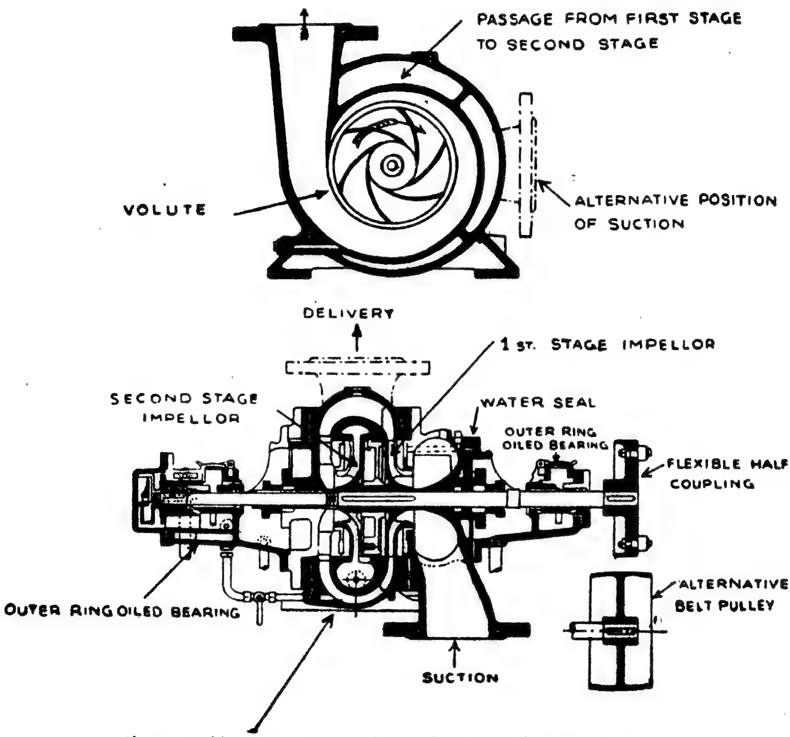


Fig. 83—Two stage medium lift Centrifugal Pump.

When the working head exceeds 100 ft., the efficiency of a single impeller pump, as is generally constructed, falls off rapidly, as either the diameter or speed or both of the impeller has to be made undesirably great, and consequently losses due to friction and eddies become considerable.

For pressures above this, therefore, centrifugal pumps with 3 to 10 impellers are arranged in series, and all built into one casing divided into separate chambers, and mounted on a common spindle. The water passes through each impeller in turn, and is directed from one to the next following and from the last into the delivery branch of the pump through guide wheels converting into pressure, without shock, the bulk of the kinetic energy transmitted to the water by the impellers. Each stage of the pump contributes equally to the total pressure. The design of the guide wheels and impeller blades is the contributing factor of the degree of efficiency attained by the pump. When water to be pumped contains grit, the leading edges of vanes in multistage pumps are liable to be twisted or worn, in which case the efficiency is seriously affected. To avoid this difficulty, the single impeller high lift pump has been designed and manufactured by Messrs. Worthington Pump Co. and is shown in Fig. 84.

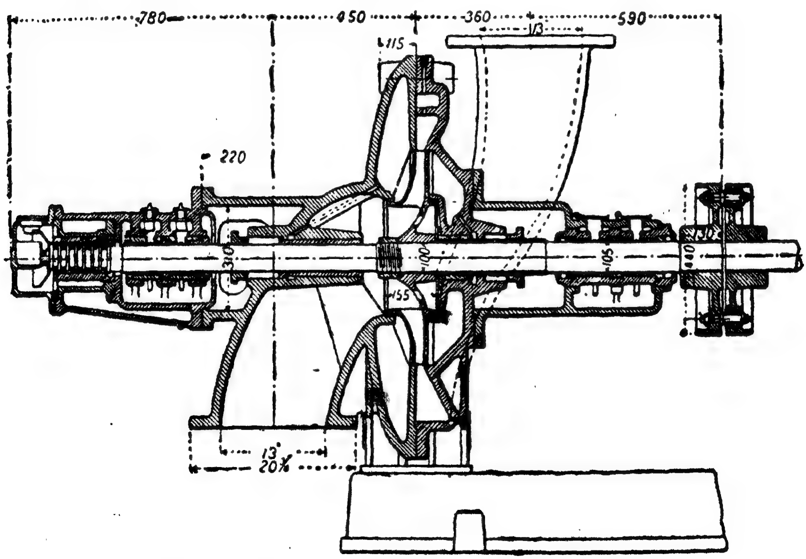


Fig. 84—Single Impeller high lift Centrifugal Pump.

The steady flow of fluid, with as few changes in velocity as possible, avoiding shocks and eddies with inherent friction loss,

is of the greatest importance. The clearance between the stationary and rotating parts in the pump must be such as to minimise back flow. The balancing of the axial thrust produced in multistage pumps is also important.

A study of the characteristics of any particular pump is to be made with a view to its selection for the load and conditions of working. The manufacturers make tests of their pumps and these data are plotted and called *pump characterisitcs* ; they may be obtained from the firm offering the pump. Among the important things to be taken into consideration for selecting a pump to work under particular working conditions are, (i) the maximum and minimum delivery of the pump, (ii) the maximum and minimum of total head against which the delivery is to be made, (iii) the maximum variations in suction and delivery heads and (iv) the nature of the drive. There is a certain range of conditions for every centrifugal pump within which it operates at its best efficiency. For a given diameter of impeller, the quantity of water discharged will vary directly as the speed and the head will vary as the square of the speed. For a given speed, the discharging capacity of the pump varies as the diameter of the impeller as stated before, and the head developed and the power required vary as the square and the cube of the impeller diameter respectively. The efficiency range of a centrifugal pump is different for different speeds and bears a relation to the speed. Impellers can be designed for working under both variable and constant rate of delivery or pressure. A great saving in power can be made by carefully selecting the unit to suit the working conditions.

The efficiencies of ordinary type of centrifugal pumps vary from 55% to 75% and increase with the capacity of the pumps.

The centrifugal pumps may be either of horizontal or vertical spindle type (Fig. 85). The latter is more expensive and requires additional accessories, such as guide, thrust bearings etc., and also the pump spindle has to be maintained truly vertical. Owing to these disadvantages, these pumps have generally shorter life and more maintenance cost. The

deterioration of vertical spindle pump is sometimes so rapid that its actual average operating efficiency is low.

The casings of the centrifugal pumps are generally made in two halves with a vertical or horizontal joint along the centre

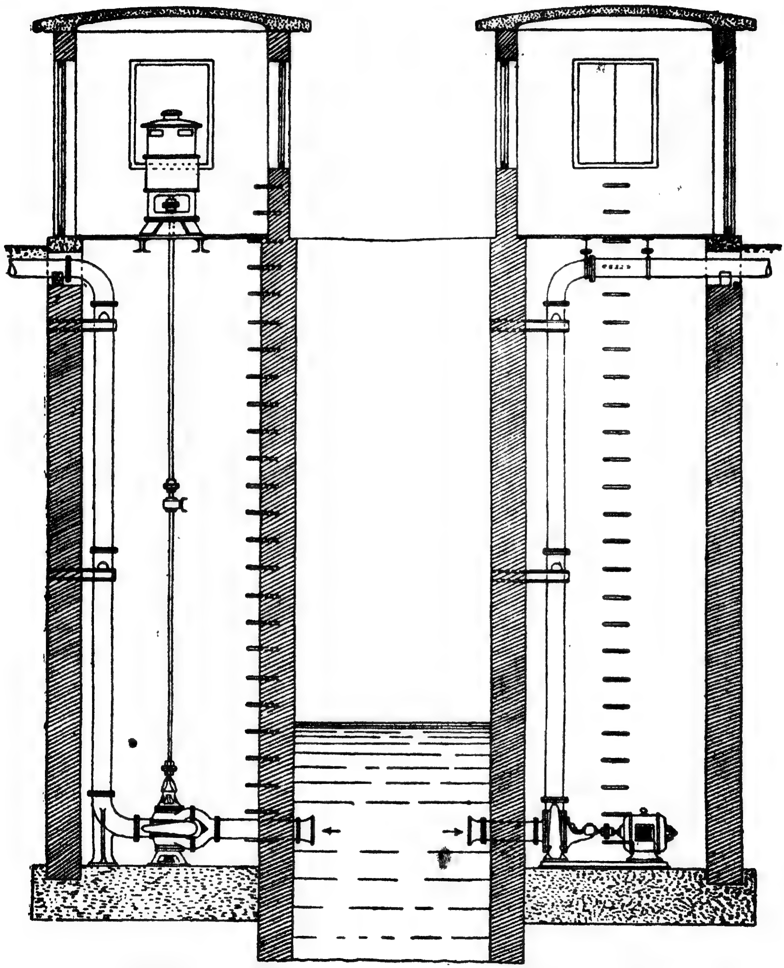


Fig. 85—Drowned Vertical and Horizontal Spindle Centrifugal Pump.

line, either of the periphery of impeller or of the spindle. The casings are generally made of cast iron and the joints are

accurately faced flanged joints. The impellers are made of cast steel, cast iron, bronze or gun metal according to requirements.

The pump spindles are made of steel, preferably stainless steel, and should be protected with removable sleeves with efficient method of lubricating bearings. The gland where the shaft enters the pump should be water sealed to avoid the leaking of air on the inlet side of the pump and consequent heavy loss of efficiency.

The manometric suction head for centrifugal pump should not exceed 18 ft., as otherwise the efficiency will be reduced.

The equipment of this type of pump is much the same as that of a reciprocating pump, only air vessels are not required except when the rising main is of long length when it is used to avoid shock on starting and stopping A non-return valve should also be fitted on the delivery side when the discharging head exceeds 100 ft The area of the suction pipe is generally made equal to the discharge area of the impeller. Suction pipe should be as short and straight as possible and should incline upwards to the pump, so that there may not be any air pocket inside it to cause interruption of flow.

Centrifugal pumps can be driven either by steam or oil engine or by an electric motor. The pump can be directly coupled to the engine or motor, or may be driven by means of belt, or bevel gear or combination of bevel or spur or worm gearing. Bevel gears cannot change speed in a ratio greater than 5 to 1, while spur gears begin to be difficult at 10 to 1 even in the double helical type. Worm gears can be used in almost any ratio but their efficiency is not good. They can work under high as well as low speed without having the disadvantage of fragile teeth.

Centrifugal pumps of axial flow type are now being largely used in pumping water from tube wells. The axial flow pump creates a smaller pressure per stage, and therefore more than one stage pump is necessary. Small impeller pumps cannot be as efficient as larger ones, for this reason sometimes the water is raised to the surface by means of an axial flow pump ; and

then on the surface and on the same shaft a second impeller known as a *booster* is fixed to raise the water to the height required.

Suction and Delivery Pipes—These should be of ample size, as straight as practicable and perfectly air-tight. Whenever bends are used, they should be of standard or as large radius as can be arranged. The suction pipe should always be fitted with a foot valve and strainer, (Fig. 79)* the area of the holes in the latter should be about twice or more than that of the pipe itself. The suction pipe should be laid on a rising grade towards the pump and the delivery pipe with as regular and uniform slope and with as few bends as circumstances permit. Where the summit of the delivery pipe line approaches the gradient line, and generally at the highest point of each vertcal bend, an air valve should be provided to discharge any air that may tend to accumulate. Such valves also serve to admit air to the pipe line to prevent the formation of vacuum in case of rapid burst or other causes at a point lower on the line. This is more important in case of large steel mains which are too thin to withstand the external

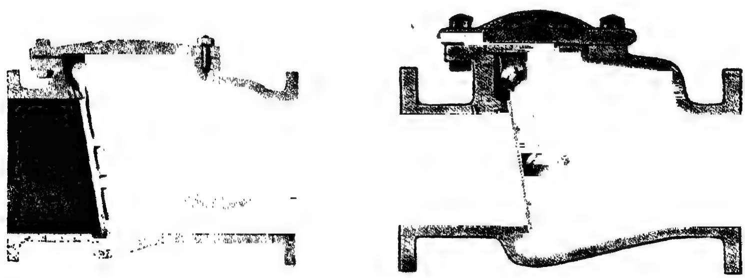


Fig. 86—Non-returning or Reflux Valve.

pressure without collapsing. A non-return or reflux valve (Fig. 86) may be fixed on the delivery pipe to prevent water from running back through the pump during sudden stoppage

* See page 190.

or repairs. It is preferable to fix a reflux valve, when it is next to the pump, with a bye-pass valve attached, as this permits of the priming of the pump. A relief valve (Fig. 87). may also be placed on the delivery pipe to save the pump from strain or fracture in case of sudden stoppage and consequent water hammer or obstruction. When two or more pumps discharge into the same main, each of them should be provided with a non-return valve and air vessel.

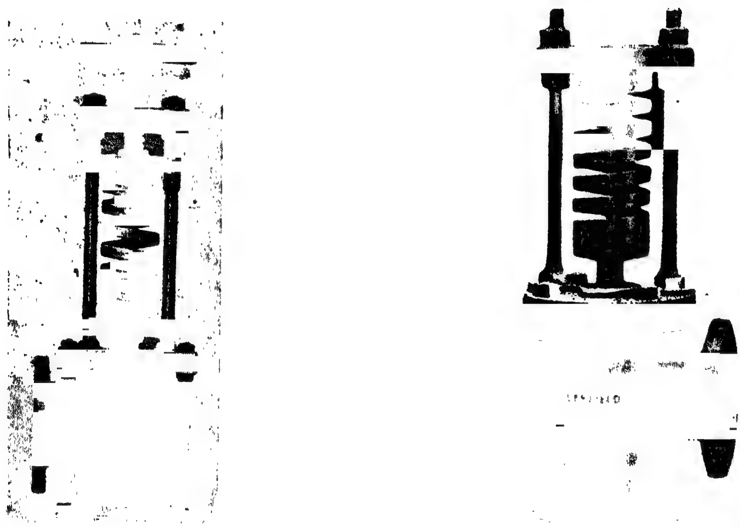
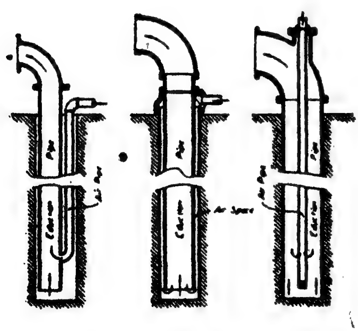


Fig. 87—Relief Valves.

Flange joints are almost universally adopted for suction pipes. An insertion rubber 1/16th inch thick is generally put in between two flanges which are tightened by means of a number of bolts. The flanges now-a-days have been standardised by the British Standards Institution and their standards are universally adopted. Sometimes in place of India-rubber insertion ring, a cord of gutta-percha is inserted between flanges and squeezed flat when bolts are tightened up. This has successfully withstood a pressure of over 600 lbs. per sq. inch.

For pipes of small diameter, galvanized iron pipes having ends screwed with screwed socketed joints are generally adopted.

Air Lift Pump—Another method of pumping, especially in the case of water from deep tube wells, is by means of air lift. It was probably invented by Carl Lischer in 1797 and fell into disuse until recently when it has been revived and improved. The apparatus consists of two parallel tubes, one for the delivery of the water to be raised and the other of smaller diameter pipe for the compressed air ; the former is called the *eduction pipe* or rising main and the latter the *air pipe.* Air from a compresser is forced through the air pipe into the submerged end of the rising main. The air bubbles rising through the water in the rising main reduce the specific gravity of the mixture and therefore the weight of the water column, so that the excess pressure at the bottom of the column due to the water pressure at the outside becomes sufficiently great to lift the mixture above the supply level and out of the top of the pipe. This excess pressure increases with the depth of submersion and must be regulated to suit the height to which the water is to be raised. In practice, the arrangement of air and eduction pipes vary very considerably. In the Pohl system, the air pipe is fixed outside the eduction pipe, (Fig. 88) in the central system, it is suspended inside the eduction pipe, while in the reservoir system, the eduction pipe is suspended in the well and air is forced through the annular space between the eduction pipe and well casing. Fig. 89 shows diagrametically a well with different terms applied to



Pohl System. Reservoir System. Central System.
Fig. 88—Arrangement of Eduction Pipe for different systems.

different parts. The design of an air lift pump requires the consideration of the following points:—

(*i*) The depth of the well, which should be such that sufficient submergence of the air pipe required for the total lift can be arranged.

(*ii*) The quantity of free air required for the water to be raised to the starting and operating pressures.

(*iii*) The sizes of air and eduction pipes and also of the compressor.

The most desirable submergence to obtain the greatest efficiency can only be determined by trial.

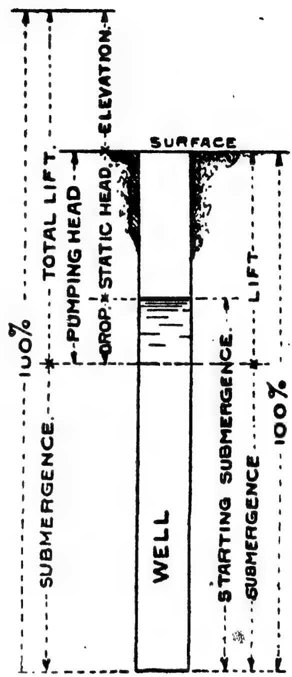


Fig. 89.—Diagram shewing different terms applied to different parts of a Well in Air Lift pumping.

In proportioning a properly balanced installation, there are two principal factors to be considered—(*i*) slippage and (*ii*) friction. As the one is reduced, the other is increased, and the proper balancing of these two elements of losses makes the most efficient installation. For efficiency, therefore, the foot piece of the air pipe should be such as to give an even distribution of air in comparatively small bubbles through the eduction pipe, so that the slippage of air is reduced to a reasonable amount and an effective emulsion is formed at a point where air enters the water. The following ratio of lift to submergence is generally adopted by American engineers.

TABLE 42

Up to 50 ft. lift	...	76% to 66%.
From 50 to 100 ft. lift	...	66% ,, 55%.
,, 100 ,, 200 ,, ,,	...	55% ,, 50%.
,, 200 ,, 300 ,, ,,	...	50% ,, 43%.
,, 300 ,, 400 ,, ,,	...	43% ,, 40%.
,, 400 ,, 500 ,, ,,	...	40% ,, 33%.

The volume of free air required to raise water is generally found by the following formula:—

$$V = \frac{h}{\log \cdot \frac{(H+34)C}{34}}$$

Where

V = Volume of air in c. ft. per gallon of water raised.
h = Lift in feet.
H = Submergence in feet.
C = A variable constant ranging from 327 for low lift and high submergence to 188 for high lift and low submergence.

In all cases, some extra allowances are to be made as the supply level of the well may drop more than the anticipated amount after continuous pumping. The starting pressure will be equal to the pressure of the column of water over the foot piece and the operating pressure, after the water in the well has reached a constant condition, will be equal to the head of water outside the well casing and above the foot piece plus the loss of head due to friction in air pipe.

The theory of pumping by compressed air is still in a nebulous state, and theoretical considerations require to be modified considerably before they can work efficiently. The following conclusions reached after an extensive series of tests at the University of Wisconsin will therefore be very helpful in designing such an installation.

1. The efficiency of air lift pumps depends primarily on flow condition in the eduction pipe.

2. Refinement in design of foot pieces is not very essential.

3. For a given submergence, a given pump will give maximum efficiency at a particular rate of pumping. With rates higher or lower than this rate, the efficiency falls. The larger the pump, the broader the range of pumping for relatively high efficiency.

4. Best efficiencies are obtained with submergencies of from 65% to 75%. For higher delivery heads, lower percentages of submergence are required.

5. There is a particular average velocity of flow for a given mixture of air and water which results in minimum friction losses. For velocities higher or lower than this particular value, the losses increase and the rate of increase is relatively great for small eduction pipes.

6. Since the air expands as it moves upward in an eduction pipe, velocities of flow increase. The use of larger sizes of pipes in the upper portion of pumps results in lower velocities. Tests show that higher rates of pumping can be secured in the high efficiency range by enlarging the upper end of the eduction pipe. Peak efficiencies were not increased over those obtainable with eduction pipes of constant diameter.

Air lift pumps have many unique features not possessed by other type of pumps, especially the deep well pumps. The absence of moving parts in wells, the possibility of working a series of wells by one compressing plant and locating the latter in the engine room under the care of the engineer, and also of installing the compressor at almost any distance from borehole, and of pumping water carrying sand and grit together with the certainty of operation, give the system obvious advantages, and in many cases outweigh the disadvantage of low efficiency.

An air lift pump with sufficient capacity can draw more water from a well than any other type of pump. Besides low efficiencies, which seldom exceed 20% to 30%, the greatest disadvantage it has, is unsuitability for shallow depth, or the well requires to be made deeper to secure adequate submergence which may not be always possible.

After a time, when the well strainers become clogged and the flow is reduced, a simple and efficient method of cleaning and increasing flow is *back blowing*. The following description of the process as given in the Journal of the American Water Works Association will be interesting.

"By closing the discharge of a well, the air pressure will force the water ahead of it back through the strainer and float the finer sand ; then by opening the discharge after a short interval, the flow of water to the surface will be resumed and the floating sand carried with it before it has time to settle. By repeating this operation, a large part of the finer material outside of, and adjacent to, a strainer may be drawn into the well and discharged at the surface and the coarser material collected around and the outside of the screen will facilitate the inflow of water. This surging back and forth of the water and sand through the strainer will clean out the openings and dislodge accumulated sands that is found to clog up strainers where the water has flowed for a long time in one direction."

Estimate of Motive Power—The actual energy that is utilised in a pumping station for raising water depends upon the efficiency of the prime-mover and the pump and the incidental losses due to generation and transmission. In estimating power for a pumping plant, these losses must be taken into consideration. The gross power required will be equal to the actual energy required divided by the combined efficiency of generation, motion, transmission, and pump which may be expressed as follows :—

$$\text{Motive power} = \frac{\text{P.H.P.}}{\text{g.m.t.p.}}$$

Where

P.H.P=The actual or Pump Horse Power.

g=Efficiency of generator, boiler, dynamo &c.

m=Efficiency of *prime mover* i.e. steam, oil, gas engines or electric motors.

t=Transmission losses such as shafting, belting, gearing or wiring.

p.=Efficiency of pump.

In this equation, the P.H.P. represents the power necessary (*i*) to raise 'G' gallons of water per minute through a static lift of 'h' in feet measured from the level of the surface of water outside the suction pipe to that of the end of the delivery pipe, (*ii*) to overcome the head 'h_1' in feet lost in friction due to the passing of water through suction and delivery pipes, and (*iii*) to maintain a head 'h_2' in feet at the end of the delivery pipe necessary for the rate of pumping. The total work thus performed is generally expressed in the following form,

$$P.H.P. = \frac{10.G.\ (h + h_1 + h_2)}{33000}$$

Where

h_1 can be found from Bazin's or Kutter's formula.

This power, however, is expressed in many other ways to suit the kind of energy employed to do the works.

The following table of conversion may be useful:—

TABLE 43

Foot lbs. per min.	Horse Power.	B. Th. U. per min.	Kilowatt.	Foot galls. per min.
1	0·0000303	0.001285	0·0000226	0·10
33,000	1	42·416	0·746	3300
44,240	1·34	56·8	1	4424
778	0·02357	1	0·01758	77·8
10	0·000303	0·01285	0·0026	1

The efficiencies of the generators, engines and pumps have already been given in the preceding pages in respective sections dealing with them. We shall now discuss the losses due to transmission of energy.

Direct Coupling —When a pump is directly coupled or connected to the prime-mover, no additional friction is involved and consequently no power is lost.

Shafting—When a pump is driven off by a counter shaft, the loss of power involved is proportional to the number and

character of bearings, the length, diameter of the shaft, lubrication and alignment.

Prof. C. J. Benjamin gives the following power absorbed by shafting in getting motion.

TABLE 44

Nature of plant.	Friction Loss in H.P.			
	100 ft. of Shafting.	100 lbs. of Shafting.	Per Bearing.	Per Belt.
Heavy Machinery	5·57	0·30	0·58	0·56
Light Machinery	2·75	0·28	0·21	0·19

The co-efficient of friction of ball or roller bearings varies from 0·001 to 0·0015. The loss in simple belt drive is usually taken to be 2% 3% of the power.

Gearing—For the purpose of calculations, the following efficiencies may be used.

Cut spur gear	0·90
Cast spur gear	0·87
Cut bevel gear	0·87
Cast bevel gear	0·84

Pneumatic Transmission—Friction losses range from 5% to 10% per mile and efficiency of motor is about 40%.

Electrical Transmission—The efficiency of an electrical plant depends upon the efficiency of motor, the line of wiring and of the transformer ; loss of power in wiring such installation in good practice varies from 5% to 6%.

Advantages and Disadvantages of Different Kinds of Pumping Plants :—

The following extracts from a paper written by Vanleer in the Journal of the American Water Works Association may be useful in comparing the advantages and disadvantages of different types of pumping plant working under certain conditions.

STEAM POWER FOR RECIPROCATING PUMPS.

Advantages.	*Disadvantages.*
Is durable. Is flexible. Will operate against a high head. High efficiency is usually obtained. Is suitable for largest pumping units.	Requires frequent adjusting. Is heavy. Occupies large floor space. Is complicated.

STEAM OR POWER DRIVEN PISTON OR PLUNGER PUMPS.

Advantages.	*Disadvantages.*
Is durable. Will be operated against a high head. Gives non-pulsating flow in multi-cylinders. Has good suction.	Is high in first cost. Is heavy. Occupies large floor space. Is slow in speed. Priming is necessary. Is not suitable for handling sandy water. Sudden stoppage may wreck the pump.

CENTRIFUGAL PUMPS.

Advantages.	*Disadvantages.*
Is relatively low in first cost. Has excellent durability. Has small weight and floor space. Is simple to operate. Starts quickly. Is suitable for electric or steam turbine drive. Is suitable for large capacity.	To produce high pressure, it requires multi-staging. Lacks flexibility for best efficincy. Must be primed at start if not submerged.

DEEP WELL TURBINE PUMPS.

Advantages.	*Disadvantages.*
Has moderate first cost. Has small size, low weight, and small floor space. Is simple to operate. Starts quickly. Has moderate speed. Has non-pulsating large flow of water. Needs no priming.	Has low durability. Is not flexible.

DEEP WELL AIR LIFT PUMPS.

Advantages.	*Disadvantages.*
Handles any kind of water. Priming is not necessary. Several wells can be pumped by one unit and at the same time. No moving part under water. Occupies little space. Will operate in a crooked hole. Will pump more water from well than any other type of pump.	First cost including compressor is high. Has low efficiency. Flow is intermittent. Requires an extra depth to the well for proper submergence.

The above list is neither complete nor conclusive, as the advantages and disadvantages of no two stations may be alike. It only gives the merits and demerits of a type of pump only in a general way, which requires other consideration.

Foundation of Machinery—The function of the foundation is two-fold. Primarily, a bed must be provided which permanently maintains the machinery firm in the position in which it is first erected without any alteration of level or alignment of any part. Secondly, it must be so designed and

constructed, that it is capable of absorbing shocks and vibrations to which the machinery is subjected, to do the full duty. To meet these conditions, it is necessary that the foundation must rest upon a solid subsoil free from organic matter and capable of supporting the maximum load coming upon it. Movement of building or structure as a whole is considered undesirable, but the movement of one part of the plant in relation to another is far more dangerous, and should be guarded against by all available means in the design and construction of foundations as it is practically impossible to arrest it or to remedy its effect afterwards.

The preliminary step in the design of a foundation is, therefore, the determination of the quality and the bearing power of the soil, and success or failure may turn entirely on it. Any mistake made in the proper proportion of the foundation or in the method of construction may cause a continuous source of trouble in operation. The following table of permissible safe load per sq. ft. of various soils may be taken as a broad guide for a provisional design.

TABLE 45

Nature of soil.	Permissible Safe Load in Tons per sq. ft.
Made ground well consolidated, black cotton soil, quicksand	0·5
Soft clay, or sand loose or wet ...	1·0
Stiff clay, compact sand or soil or loam dry or moist	2·0
Red earth	3·0
Compact gravel or coarse sand clay ...	4
Soft rock	5
Ordinary rock	10
Hard rock	15
Gravel and coarse sand well cemented ...	6

In addition to the vertical loading, the horizontal thrust, overturning and sliding actions must also be provided for in designing a foundation. Rock will bear safely very heavy

machinery, but it has one failing, that is of transmitting vibration along a vein to a great distance or sliding down along the line of cleavage. Cases are not rare where failure of foundation has been attributed to such causes. Machinery should not be placed direct on hard rock which is extremely rigid and consequently cannot absorb shock or vibration. To avoid this difficulty, the machinery may be founded on a layer of concrete over a layer of 5 or 6 inches of fine sand over the bed of rock. Rubber and felt have also been used, but they are more expensive and the results obtained are somewhat uncertain. When the foundation is on a made-up soil or a soil containing organic matter, it is best to carry the excavation for it down below the firm soil, if available within a short distance from it, or to rest the entire foundation on piles. An alternative to piling, where hard bottom can be found at reasonable depth and the top soil is bad but not extremely so, is to excavate the surface soil down to hard soil and fill in with carefully rammed sand enclosed on all sides by sheet piling. The more usual practice however is to build a raft or platform of re-inforced concrete which distributes the pressure over a wider area. This raft should be so designed as not only to bear the load of the machinery but also the shocks transmitted to it.

For the general design of the foundation, the position of bolt holes, pipe trenches or cable ducts are always given by the manufacturers. The engineer is to see that the foundation below is sufficiently solid to carry the weight of the masonry base of the plant provided by the manufacturers plus the weights and pressures transmitted by the machinery.

The foundation is made generally of cement concrete, re-inforced or otherwise, the proportion commonly used is 1 part of cement, 2 parts of sand and 3 parts of ballast. In Bengal Rajmahal stone and Waria sand or crushed quartz is usually used for aggregate matrix. In laying out the foundation, care should be taken to set out accurately the position and alignments of pipe trenches, cable ducts, bolt holes &c. When the machinery is placed in position and level, the bolt holes are grouted and the whole of the concrete base including the sides,

pipe trenches &c. are plastered with cement. Grouting, whether of the bed plate or foundation, is an operation that should be very carefully carried out. A good deal depends on the correct preparation of the grout, but it is even more important that it should be properly applied, and, so far as the bed plate is concerned, this is not always an easy matter. The bed plate is generally levelled up by means of machined iron wedges of ample area and thickness placed in pairs, one on top of the other, in such a manner that when the upper one is driven it raises the bed plate. The portion between this and the foundation below is filled with grout, and upon this grout and the wedges the engine rests in a proper condition of level. Care should be taken that no oil or other materials interfering with the setting or strength of cement get mixed with grout, as these create endless trouble during the operation and necessitate doing the whole work over again. The grouting of holding down bolts is a comparatively simple matter. It is only necessary to stir the grout well when it is being poured, and thus prevent the formation of air or water pockets.

The material most widely used for grouting are equal parts of portland cement and sand well mixed with water.

The subject of machinery vibration is an extremely complicated one and requires very careful study. So many factors contribute to the production of vibration that closest inspection and careful experiment are necessary to locate the cause of such trouble. It may be due to the inherent defect in the design or construction of the machinery itself or to the defective foundation. The transmission of vibration through the soil is a matter beset by much complexity. An oscillating foundation propagates vibration waves, which diminish with the length of their travels from the centre of origin. Owing to variable condition of the soil, it may be quite possible that these vibration waves may travel more in one direction than another and also more in one spot than another. There was a case in Calcutta where the vibration waves from an oil engine driving an oil mill were perceptible in an adjoining property about

100 ft. away from the engine, and the mill had to be closed owing to the threatened litigation of the property-owner.

With the object of isolating a foundation from the surrounding soil and absorbing vibration and shock, various methods have been employed in Western countries and America but it is not within the scope of this book to deal with the details of such methods. We shall simply allude to them.

Besides the well known expedient of placing layers of ordinary felt and sheet lead under foundation, four other systems have been used in different places, viz., (i) *Harburg system,* (ii) *Prache system,* (iii) *Korfund system,* and (iv) *Mascolite system.* In the first system which is generally used for heavy machinery, rubber sheet of special quality is inserted under the foundation or footings of the machinery. A thickness of 3 mm. to 4 mm. is generally used for machinery weighing 3 tons, but for heavier machinery thicker sheets are necessary. This is generally used in Germany. In England, the Prache system is used to a considerable extent, under similar conditions. In this system, the foundation raft of the machinery rests on a number of circular india-rubber stools which when under compression are 4″ in diameter and 3 inches thick, these stools in turn rest upon the ordinary foundation. To enable ready inspection or renewal of a stool, a trench is provided all round the raft. The rubber employed is of special quality and is specially treated in order to lengthen its life. The third system consists of plates built up of single cork strips specially selected, treated and impregnated. These strips are securely bound together by an iron frame with internal struts dividing the plates into sections. These not being level with its surface do not carry any weight.

The particular feature of the last system is the special proofed felt used either alone or in combination with cork and rubber. The felt employed is a special mixture of fibres which has been selected after lengthy trials as the best sound and vibration absorber. It is claimed that the type of felt used in this system does not lose its resiliency, and consequently there

is no permanent set under the worst conditions obtaining in practice.

Specification of Plant—When specifying machinery for a water-works, the water-works engineer should not attempt to go into the details of pumps or engines to be used but confine himself to generalities, such as quality of materials to be used in construction, general arrangement of parts as well as the plant. Otherwise, he is likely to handicap the manufacturer in putting forward a tender for the most efficient machinery that can be obtained. The specification should, however, give the full details of the duty to be performed, the lowest suction and highest delivery levels, the diameter and length of the rising and suction mains, the character of water to be pumped, the arrangement of intake, the maximum and minimum rate of delivery, if it is variable, and any other information that may be helpful to the manufacturer. Most manufacturers can offer machinery for water-works installation, but it is for the engineer to word his specification in such a way as to permit broad competition on suitable equipment for the required condition. In this connection, only such equipment is to be specified, which is found fully reliable in its performance elsewhere.

The relative values of different tenders are compared on the basis of annual maintenance charges. The annual charge is found by adding up the annual cost of fuel, the cost of lubricating oil, the cost of spares, the stores and the pay of the staff required to work the plant, also the annuity for sinking fund for the cost of machinery in different tenders, that of the building to house them.

In scrutinizing tenders for such a plant, the speed of the different parts of the machinery, and the soundness of the working parts, and also the reputation of the manufacturers should all be taken into consideration. The slower the speed, the more substantially the working parts are built, and usually the more durable the plant will be found to be.

A choice can then be made from the tenders submitted, but the young engineer should remember that in selecting plant price is not everything. As a rule, however, if the tenderers

for the plant are well-known manufacturers, one is fairly safe in selecting the plant which gives the lowest working costs.

Engineers should particularly guard against cheapness for cheapness' sake, for something must be left out after a certain point if costs of manufacture have to be brought down. After all, the manufacturers cannot afford to give away a good class of material, skilled workmanship and scientific design for nothing.

Moreover, the engineer himself must safeguard his professional reputation, and to the young engineer starting out on his profession he would do well to bear in mind the joke about engineers and doctors by a well-known engineer, who was comparing the two professions. "The doctor", he said, "may bury his mistakes, but the engineer cannot do so. It stands always as a monument of his failure."

Buy well and insist upon getting what you pay for.

Simplicity in design is an important factor in the selection of a tender. The simpler the machine is in general construction, the easier it is for the operator to work and to give proper care and attention to the different wearing parts. Complicated machinery always requires a higher class, and consequently more costly labour for operation and repairs. Complications often lead to accidents, as it is very difficult to get trained labour in this country.

Clauses for testing machinery and penalty for failure of performing the required duty with the fuel stipulated in the tender are also incorporated in the specification to prevent unscrupulous makers attempting to defeat the object of open competition.

In case of failure of duty, the capitalized value of the extra annual cost over that specified is generally realised as penalty.

Each pumping plant should have a standby, and each station should be provided with a small workshop and sufficient spare parts to meet any contingency.

Testing of Pumping Plant—The testing of a plant is carried out with the object of ascertaining if under certain

specific conditions the plant performs the duty for which it is required, and to see that the maker has carried out his part of the transaction according to the conditions laid down in the specification. The duty of the plant is generally specified in terms of the number of gallons of water to be pumped against a specified suction and delivery head, including the head lost in friction and other causes for the flow of water through the pipes. The maker generally guarantees that the plant offered will be capable of performing the duty specified with a stipulated amount of fuel, or steam or electrical energy per P.H.P. hour.

The scope of the experiment is to determine the power of the prime-mover as well as the pump to perform the work specified with the stipulated quantity of energy.

For determining the capacity of the pump, the following operations are generally necessary.

(*i*) Measurement of the quantity of water delivered during different periods of test.

(*ii*) Measurement of the suction and delivery heads during the same periods of test.

The first operation may be accomplished by a water meter fixed on the delivery of the pump and taking readings at equal intervals during test. Or it can be done by taking water levels in a reservoir in which water is delivered and calculating the quantity from the depth of increase or decrease during different intervals of test and the area of the tank filled.

For the second operation, gauges are fixed on suction and delivery pipes and readings are taken at equal intervals.

From the observations, the pump horse power or the power actually utilised in pumping the water can be calculated by the common formula :—

$$\text{P.H.P} = \frac{\text{Gallons of water pumped per min.} \times 10 \times \text{head lifted.}}{33{,}000}$$

In ascertaining the capacity of the pump, the same is to be run with such speed as to deliver the quantity of water specified and at the same time the speed of the pump is observed by means of a speedometer or other suitable instrument to compare with it with that guaranteed by the maker.

The testing of prime-mover is more complex, and many points in the construction and theory of the motor or engine have to be settled by a series of experiments. Here we shall allude only to those tests which a water-works engineer in this country is called upon to perform.

Steam Engine—There are two ways in which a steam pumping plant is specified in this country. The first is that the plant will be capable of performing the duty specified with certain amount of fuel of certain calorific value per pump horse power hour and in the second, it is stipulated that a consumption of steam per pump horse power hour will not exceed a certain amount. In the former case, the supplier guarantees the efficiency of the boiler in addition to that of the engine, while in the latter case, only the efficiency of the engine is guaranteed.

In the first case, it is necessary to measure the quantity of coal burnt in the boiler during different periods of pumping. To do so, the coal is first weighed into bags of 50 lbs., 100 lbs. or 200 lbs. nett according to the size of the boiler, and are stored near the firing floor. As the coal is used, the bags should be folded and stacked in a convenient place where they can be quickly counted. The time for beginning to fire each quantity should be noted.

In starting and stopping, great care is needed or considerable inaccuracy may be introduced, especially when the duration of test is short. From half-an-hour to one-hour before the test is commenced, the fires should be cleaned and made up, a feed pump being kept going to maintain the water supply. It is desirable that the level of the water in the boiler gauge glass should be brought up as nearly as possible to some pre-determined datum before starting the trial. After the fires have burnt low, they are carefully levelled and the pressure gauge is watched. It will be observed that the pressure will begin to decrease rapidly, and at this moment the trial should commence. The first charge of coal is then fired, and immediately after the ashes and clinkers if any, are removed. Near the end of the trial the fires are again levelled and the pressure

gauge watched for the decline of pressure at the instant. When the pressure is declining and the condition of the fire is almost the same as in the beginning, the trial should terminate.

During the test, small samples of coal should be collected from each bag for the purpose of analysis and ascertaining the calorific value. The samples so collected are weighed at the end, and the whole weight divided by the time of trial is subtracted from the rate at which the coal is consumed.

In testing boilers, the records of fuel, feed water, temperature, and pressure are to be kept for different periods of test. The measurement of feed water is best effected by weighing it and failing that, a tank of known dimensions is to be used.

Where the rate of steam consumption per P.H.P. hour is specified, the steam consumption is measured by weighing the air pump discharge from condenser which, instead of delivering into the hot well, is arranged to deliver into a measuring tank, or it may be measured over a weir. Although this method is most satisfactory, it is not always possible, and in that case, the only alternative is to measure the feed water into the boiler and use the latter for nothing else than to supply steam to the engine. To enable this to be done with confidence, the feed pump should be stopped just before the test starts. The stopping of the pump will cause the water in the gauge to fall, and when the level in the gauge glass has reached the pre-arranged level, the pump should be re-started and the test begun. The feed pump is to be worked at an uniform rate until near the end of the test, when it is again stopped and the water level is brought back to the pre-arranged level. At the moment the water level reaches the mark again, the time should be noted. Thus, an accurate measurement of the feed water and consequently of the steam consumed will be obtained.

Oil Engine—In case of an oil engine prime-mover, the water-works engineer is generally required to determine the quantity of fuel consumed in performing the duty specified. For this purpose, the engine is run to the stipulated speed, and

while working under that speed, the consumption of fuel is determined. The speed can be measured at intervals by means of a hand counter and stop watch or by a Tachometer. The fuel can be measured either by placing a tank containing the fuel on the platform of a weighing machine and connecting the same to the engine with a flexible metallic tubing ; or its volume can be measured by ascertaining the rate of fall of the level of the liquid in the tank. The tank should be closed with a pet cock for equalising the pressure inside the tank while the fuel is being used. A gauge glass may be fixed on the side of the tank to indicate the level of fuel in the tank during different periods of the trial.

Electric Motor—In this case, the energy consumed while doing the specified duty at a stipulated speed is measured by observing the readings in the voltmeter and ammeter and watt hour meters during different periods of the trial. The temperature rise should also be measured, and this should be done in the manner specified in the British Standard Specification for the type of machine, and must not exceed the permissible rise stated therein. The speed can be measured in the same way as in the case of an oil engine.

Measuring Instruments—The accuracy of the measuring instruments, such as gauges, meters, &c. used in the testing of a plant, should be tested beforehand. The test can be made by comparison with standard instruments or in the Government test house, where every arrangement is available for accurate testing.

CHAPTER IX.

CLEAR WATER AND SERVICE RESERVOIRS.

Clear Water Reservoir—Although these two classes of reservoirs are used for different purposes, both of them may be termed "balancing reservoirs or tanks". The clear water reservoir is used to collect the water from filters and to store it until it is pumped into service reservoirs or distribution mains, as the case may be, for delivery to the consumers. They are generally built below ground and must have a top water level somewhat lower than the outlet wells of the filters. The capacity of this kind of tank varies with the aggregate filtering capacity of the different filters, the rate and period of pumping. As stated before, the number and size of filters in a slow sand purification plant should be made such that under all conditions of working they can easily cope with the maximum daily supply. In such cases, the filter delivers water in a more or less uniform rate, but the pumps may be required to draw water from clear water well at considerable fluctuating rate ranging from nothing at certain hours of the day to several times the total average rate of filtration at others. Hence, it is necessary to provide for storage to enable the excess water filtered to be collected for balancing the rate of pumping. When the pumps are worked continuously for 24 hours at an uniform rate, the capacity of these tanks need not be large. The clear water reservoir for Howrah water-works was designed to hold only 15 minutes flow from the filters. But when the pumping is limited to 8 or 10 hours a day, the capacity should never be less than 16 hours flow from filters. For determination of the approximate capacity of these tanks, the following formula in the case of slow sand filters is suggested:—

$$C = (R - r)D$$

Where

C=capacity of the tank in gallons.

D=maximum daily hours of pumping.

R=rate of pumping in gallons per hour.

r=total rate of filtration per hour.

In case of purification by mechanical filters, the sizes of filters are usually made such that the total delivery from them

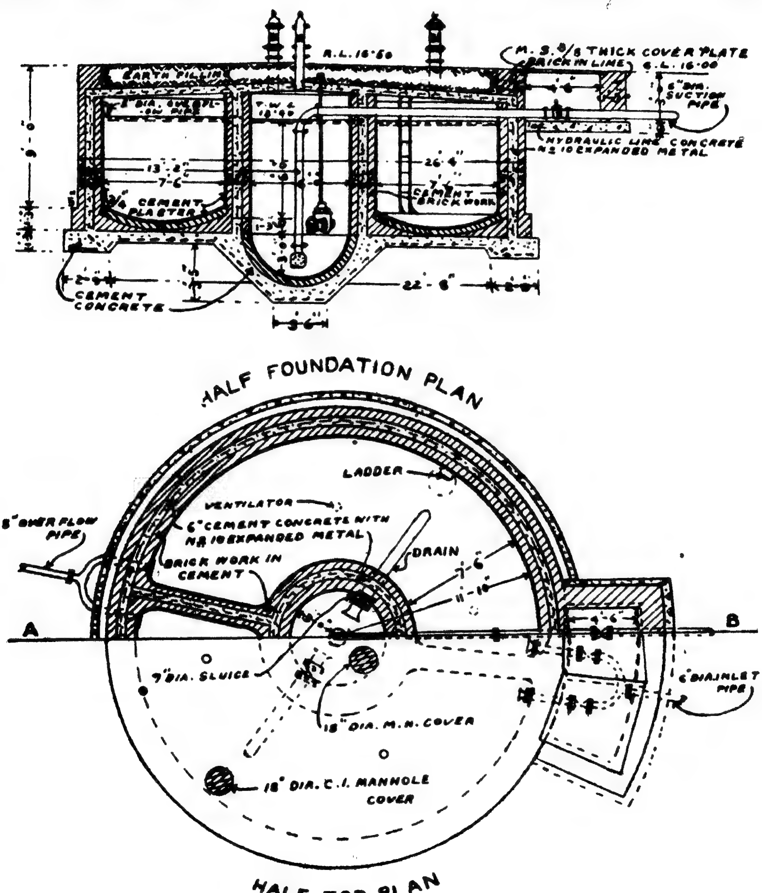


Fig. 90—Clear Water Reservoir.

is equal to the rate of pumping ; consequently, the capacity of a clear water tank need be very small.

The type of design usually adopted for such reservoirs in Bengal (Fig. 90) is of reinforced concrete lined both sides with 5 inches brick and generally circular in form. A brickwork lining is generally made in place of falsework, which is generally expensive and of poor workmanship in this country. The one objection to this method of design is that the cement plaster cannot be applied direct to the concrete which, on account of this, is consequently likely to be left in many places in a honeycombed state.

The other type is constructed entirely of brickwork of sufficient thickness to retain the soil round it and is generally made rectangular in form. These reservoirs must be water-tight under all conditions of working to prevent subsoil pollution. This requirement is usually fulfilled by the application of cement plaster ¾″ thick with suitable water proofing composition like *pudlow, ironite* or *sika.* Waterproof surface can also be obtained by *sylvaster process.*

A covering over these reservoirs is essential for protection against dust, smoke, fumes and other form of surface pollution. The water in a covered reservoir is maintained at more equable temperature and is less liable to algeous growth owing to exclusion of light. The roofs are made of reinforced brickwork, reinforced concrete, or of brick arches supported by series of brick or reinforced concrete pillars standing on the floor of the reservoir. The roof is generally covered with 2 or 3 ft. of earth and sloped off to drain into a surface drain leading outside. The tanks are usually made 8 or 10 ft. in depth according to the condition of the subsoil.

Provision of the following accessories is also necessary for the satisfactory working:—ventilating cowls, manhole and ladder for access and cleaning, and a water level indicator. (Fig. 91). The reservoirs are sometimes made in two sections so that one can be used independently of the other at the time of repairs.

The cost of these tanks vary from Rs. 15 to Rs. 30 per 1000 gallons when constructed of reinforced concrete. In

alluvial soil of Bengal when made deeper than 8 or 10 ft., their construction has been found to be very difficult and expensive owing to the laying of the foundation over the layer of running sand present almost in every district especially on the left side of the river Hooghly.

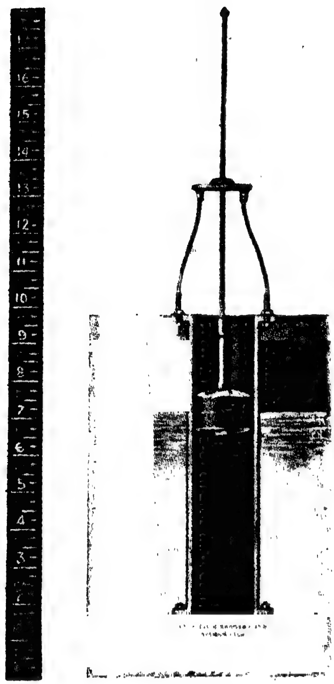


Fig. 91.—Water Level Indicator.

Service Reservoirs—These have two distinct functions to perform, viz., (*i*) to facilitate distribution by equalising pressure ; (*ii*) to furnish storage to meet the variable demand during different hours of the day. When no such works are constructed, the pumps are required to be worked continuously at fluctuating rates and are subjected to serve strain, especially when several taps are suddenly turned on, or a pipe bursts. Other means besides the provision of service tanks have been adopted elsewhere to overcome such shocks on the pumps. In some small direct pumping waterworks in Europe, accumulators have been employed. They consist of heavily weighed rams working inside strong vertical pipes with stuffing boxes at the top. The vertical pipe is connected with the force main, and any marked variation of pressure is taken up by the vertical movement of the ram. This arrangement is not very sensitive and may be useful when the fluctuations are not great. In Germany, large air chambers are frequently used in place of service tanks for equalising pressure on pumps. These chambers are filled with air at the time of ordinary pressure in the main. During a rise in pressure from the ordinary amount, the chamber takes in

a certain quantity of water and the volume of air is reduced by that amount by compression, and when the demand is suddenly increased, by the drawing of water from the distribution mains, the expansive force of the compressed air promptly returns the water taken into the chamber, and thus equalise the pressure while the engines are slowing down. Accumulators and air chambers do not afford storage and are necessarily poor substitutes for service tanks.

The capacity of these service tanks should be sufficient not only to meet an exceptional demand over the ordinary rate of pumping but also it should be capable of maintaining the supply when the rising main, engine or pump is under repair, and should any accident happen to the installation involving temporary stoppage. The capacity depends on the hours and rate of pumping, the fluctuation of demand, and lastly on the system of supply, i.e., whether it is continuous or intermittent.

The practice in Bengal is to design these reservoirs to hold a third of the average daily supply and this has been found to be fairly satisfactory. In some cases where the fluctuations of demand are great in comparison with the capacity of the pumps, it may be necessary to make the capacity equal to half of the average daily supply. Where requisite data is available, the capacity can be worked out in a way similar to that described for storage or impounding reservoirs, the difference being that these tanks will maintain the balance between the supply from the pumps and the draw off from the distribution main, instead of that between the rainfall and the daily demand. Where no natural site can be had for a service reservoir to be built at sufficient elevation to give the requisite statical head for a satisfactory pressure in the distribution pipes, it becomes necessary either to erect a stand pipe or a service reservoir supported on a tower. A standpipe is a tall pipe of small diameter used as a makeshift arrangement instead of a service tank, though it is more useful when it is made of a large diameter to provide for storage. It was once largely used in America ; in Bengal, a service tank is generally adopted in

its place. The height of these tanks should be so fixed that the capitalised cost of pumping and initial cost of pumping machinery plus the cost of distribution system become a minimum. The extra cost of the tank due to a small increase of height being not much may be neglected. With the increase of height of reservoir the cost of pumping and initial cost of pumping machinery increases, while the cost of the distribution system correspondingly decreases owing to the decrease in the sizes of pipes. The determination of an economical height of tank is a laborious process and is a matter of experience and judgment on the part of the designer. In all cases, however, the height of these tanks should be at least such as to give a head which permits of hydrants and connections to houses and street standposts having the desired quantity of water at the requisite pressure. Service Reservoirs in Bengal are generally made 30 to 60 ft. high with the exception of the Talla tank for Calcutta Waterworks which is 120 ft. high. The tanks in this province are usually made 10 ft. deep.

Four types of construction are usually used in this country viz. :—

(i) reinforced concrete tank on brick or reinforced concrete tower ;

(ii) steel tanks, either circular or rectangular, on brick pedestal ;

(iii) steel tanks on steel staging ;

(iv) pressed steel tanks on steel staging.

Having thus decided upon the size and height of the tank, its general design can be proceeded with. The design of a masonry tower or a steel staging is a matter of general engineering and need not be discussed here.

CIRCULAR TANKS—The fundamental principles, on which the design of these tanks should be based, are comparatively simple and do not require elaborate explanation. When a circular tank is filled with water, it is subject to a bursting pressure due to the pressure of water within it. The determination of stresses in the shell of a tank is a simple problem. The

following formula is frequently used in determining the thickness of steel shell :—

$$t = \frac{62 \cdot 5 \text{ h d}}{12 \times 2 \text{ s. e.}} + \frac{1}{4} = \frac{2 \cdot 6 \text{ h d}}{\text{s. e.}} + \frac{1}{4}$$

where t—the required thickness at h. ft. below top water level.

h—height of water in ft. above the section.

d—diameter in feet.

s—allowable stress of metal in tons per sq. inch.

e—the efficiency of the riveted joints.

The value of e depends upon the size and spacing of rivets, but is generally taken to be 0·50 to 0·60 for single and 0·65 to 0·75 for double riveting. The value of s·e· is frequently assumed to be 10,000 lbs. per sq. inch. The thickness of the plate should never be made less than ¼ inch.

The bottoms of the tanks are made either flat, conical, hemispherical or a segment of a sphere. In Bengal, they are usually flat, and in some rare cases, have been made hemispherical. Recently, a hemispherical bottomed tank on brick tower has been designed and constructed by Mr. G. B. Williams for Cooch Bihar Waterworks. This tank furnishes a good example of architectural beauty combined with economy.

The thickness of a spherical bottom is computed by the application of the well known theorem of Mr. Rankine that the intensity of tension in spherical shell is equal to one half of that due to the hydrostatic pressure with a cylindrical shell of the same diameter. Hence, the thickness of the shell can be arrived at by the following formula :—

$$t = \frac{1 \cdot 3 \text{ h. d.}}{\text{s.e.}} + \frac{1}{4}$$

The bottom plates when made hemispherical have to be shaped in dies. For circular seams lap joints and for radial seams, butt joints with cover plates have been found to be quite satisfactory. In these joints the diameter of the rivets are usually made about twice the thickness of the plates, and rivets of less than 5/8 inch diameter are seldom used. The pitch of

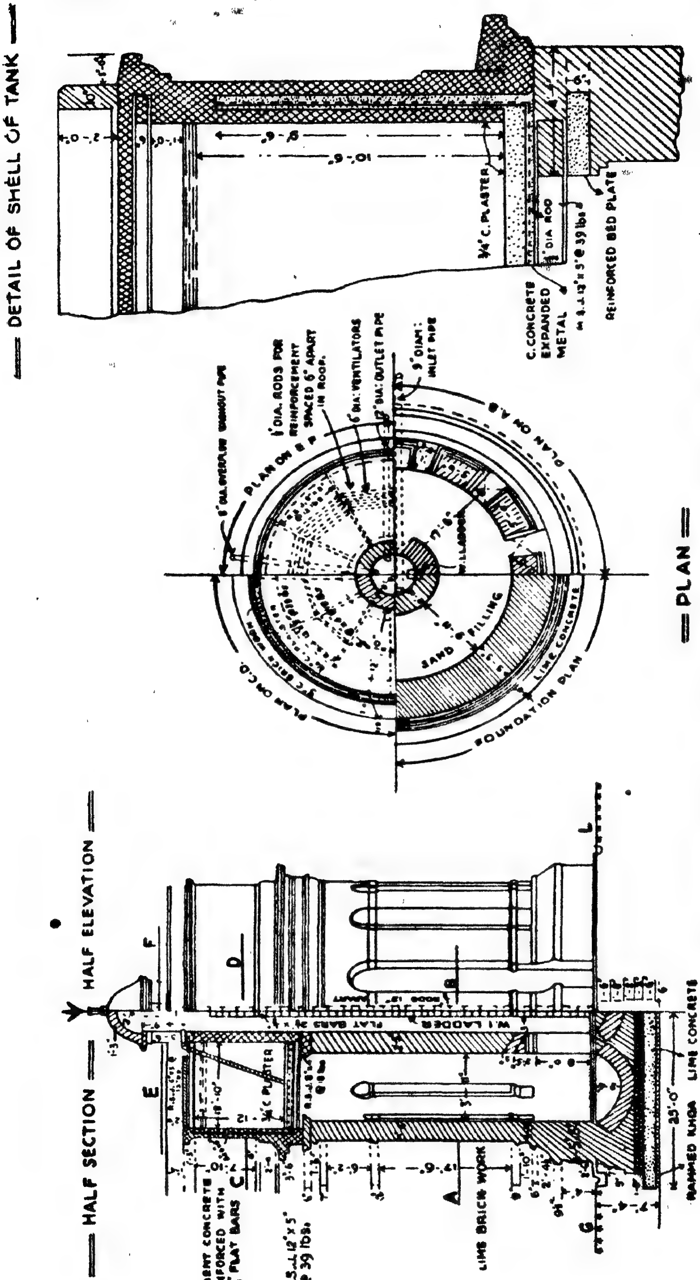


Fig. 92—Elevated Service Reservoir, Mymensingh Water Works.

rivets are generally made three times the diameter. The riveting should be done in such a manner that the new heads are formed from the opposite side of the plate to which caulking is done.

All plates should be caulked from inside with round nosed tools. Plates up to ¾ inch thick are generally punched and any plate thicker than that is usually drilled.

In case of a reinforced concrete tank (Fig. 92), the common practice has been to make the horizontal steel ring of reinforcements sufficiently strong to take up the whole of the hoop tension of the cylinder as represented by the formula :—rwh, neglecting the tensile strength of the concrete and thus ensuring the watertightness of the tank. The spacings of horizontal rings and their sectional area have been varied in the vertical according to the decrease of stress from bottom to top, but for some distance from the top, the maximum spacing with minimum sectional area of the rings is maintained. Chart III gives the sectional area of steel required for different diameters of tank at different depths of water level from the top. The thickness of the shell is made sufficient to encase all steel rings thoroughly and to be watertight under pressure. A 9 inches thick shell has been found to be quite ample for a ten feet deep tank.

Vertical reinforcements of steel have been used to support the horizontal rings and to take up shear and other stresses in the vertical plane. Another important point, which requires to be carefully considered, is the disposition of the reinforcement near the base of the tank. Here, the tendency of the shell to increase in diameter owing to bursting pressure of water causes a crack at the corner where the shell meets the floor. The amount of stresses at this point is very difficult to estimate, and consequently, the designer has to depend upon his previous experience or upon the experience of others. In Bengal additional vertical and bent rods are usually provided in this

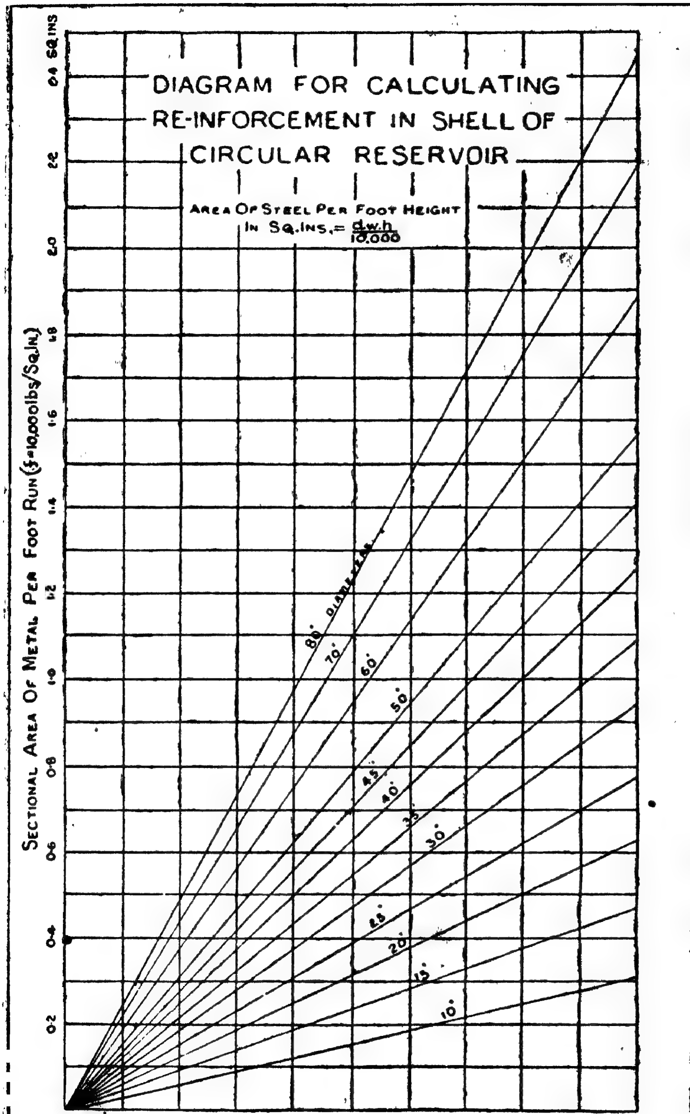


Chart III.

Note :—d = diameter of tank in feet = 2r.
w = weight of water per cft. in lbs.
h = height of tank in feet.

junction extending a few feet up into the wall and the floor for these special stresses.

To make the concrete thoroughly watertight, the selection and gradation of aggregates should be such that maximum density is secured, a sufficient amount of mortar with a liberal proportion of cement should be allowed. The concrete should be thoroughly mixed and laid sufficciently wet, so that it can flow into, and fill in, all spaces without forming honeycombs. Only such cement should be used as possesses those properties specified in the British Standard Specification, and in addition develops the minimum temperature during setting.

In Bengal, a false work for circular shells has been found to be expensive, for which reason, the concrete is usually laid between two rings of 5″ brickwork in cement mortar. This has been found to be very cheap and satisfactory in many places. In this method of construction, the plaster cannot be applied direct to the concrete which is often left in a honeycombed state. But this can be avoided by constructing the inner ring of brickwork in mud mortar, for laying the concrete. Subsequently, the temporary ring of brickwork can be removed and plaster applied direct to the surface of the concrete to make it perfectly watertight. This method of construction has worked very satisfactorily in several works recently constructed.

The bottoms of both steel and reinforced concrete tanks are generally made flat supported by a number of joists spaced 2·5 to 3 ft. apart. The floor is designed as a slab or beam to span over this distance. The floor may be supported by a circular girder resting on a framework of 4, 6, 8 or 12 steel pillars, or on a circular tower of brick or masonry.

Mr. J. N. Hazlehurst gives the following table for the stresses in a circular girder in his book on "Towers and Tanks for Waterworks", where w=total weight to be supported ; r= radius of the beam.

TABLE 46.

No. of points of support.	Reaction at point of support.	Max. shear.	B. M. over point of support.	B. M. between points of support.	Max. Torsional Moment.
4	$\frac{w}{4}$	$\frac{w}{8}$	—·0315 wr	+ ·01762 wr	·0053 wr
6	$\frac{w}{6}$	$\frac{w}{12}$	—·01482 wr	+ ·00751 wr	·00151 wr
8	$\frac{w}{8}$	$\frac{w}{16}$	—·00827 wr	+ ·00416 wr	·00063 wr
12	$\frac{w}{12}$	$\frac{w}{24}$	—·00365 wr	+ ·00190 wr	·000185 wr

The total weight on beams and floor are generally increased by 33 per cent. to allow for the constantly varying nature of loading without increasing the factor of safety.

Wind pressure should also be carefully considered in connection with the designs of towers as well as tanks. This is specially important, as several failures have occurred in America due to the insufficient provision for such stresses.

Theoretically, the total pressure on a cylindrical tower or tank is equal to $\frac{P.D}{2}$, where P=pressure of wind and D= diameter of the cylinder. This calculation does not however take into account the negative pressure to which a body is subjected when exposed to a current of fluid. Several experiments have been made both in England and Germany by eminent authorities like Prof. Unwin, Berthon and others, but the results so far obtained are not very conclusive, and further investigations are necessary to arrive at a definite result. Engineers both in America and England, however, generally assume the total pressure for square bodies=P.D. and for circular ones $\frac{P.D.}{2}$ to be fairly correct. The German Govt. however prescribes the following formula:—

For circular structures	..	0·67 P.D.
,, Octagonal ,,	..	0·71 P.D.
,, Square ,,	..	P.D.

These structures should be so designed that they can resist a wind pressure of not less than 30 lb. per sq. inch of the vertical surface with a factor of safety of 4 in case of iron and steel structures and co-efficient of stability of 2 against the action of gravity. A structure is on the point of overturning when the moment of the resultant of wind pressure just exceeds the overturning moment of the weight of the structure. In case of circular or rectangular tanks, the moment of the wind pressure is equal to $\frac{PDH}{4}$ and $\frac{PDH}{2}$, where P = the pressure of wind, D = diameter or length of side of the tank and H is the height of the tank, as is usually the case in all fluid pressure. The overturning moment of the structure is equal to the total of the weight of the structure multiplied by the distance from its centre of gravity to outer edge of the base. When the former exceeds the latter, anchorage is necessary. This is possible only in case of a steel structure. In case of masonry towers this consideration is hardly necessary, as they are generally sufficiently massive to withstand any storm.

In determining wind stresses in steel staging, the tower is considered as a cantilever fixed at the lower end. The method of analysis suggested by Prof. Meston in a paper published in Engineering News in 1898 is as follows:—

"If a horizontal section is taken on the top of each bay of the tower cutting only the pillars, we can get the vertical component of the post stresses as in a beam made up of posts. Fig. 93 shows such a section composed of 8 posts. If A represents the area of a post, R radius of the tower at the section. The maximum stress in the post A will occur when the wind blows at right angles to Mm_1. If M be equal to the moment of wind pressure about the section, then the stress "f" in the post will be equal to $\frac{MR}{I}$ where I is the moment of inertia of the section about Mm_1, which in this case is equal to $4AR^2$.

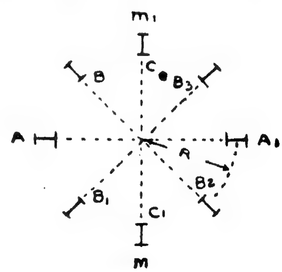


Fig. 93.

If the moment of inertia of the posts is neglected, then

$$f = \frac{MR}{4AR^2} = \frac{M}{4AR}$$

or the total stress $= \frac{M}{4AR} \times A = \frac{M}{4R}$

In a six post tower $I = 3AR^2$

and the maximum stress $= \frac{M}{3R}$

In this way the vertical load on the top and bottom of each bay can be readily calculated. The stresses in an inclined post are equal to the product of the vertical load and the secant of the angle of inclination of the post from the vertical.

The foundation of a steel or masonry tower should be on an absolutely solid ground, and should be sufficiently broad to bring the intensity of load within the safe limit of the bearing capacity of the soil. In good soil, lime concrete of 2 to 3 ft. depth has been found to be quite satisfactory. In bad soil, such as running sand, the reservoir is generally constructed over a reinforced concrete raft resting on a number of piles, the number being determined in accordance with the bearing power of the soil and the load of the structure. Each post should rest on and be anchored to a solid masonry foundation.

RECTANGULAR TANKS—These may be of the following types :—

(i) Riveted steel tanks.

(ii) Pressed steel tanks built up of standard section bolted together.

(iii) Reinforced concrete tanks.

(iv) Cast Iron tanks composed of standard sections bolted together.

Cast Iron tanks are generally 3 to 4 times as heavy as wrought iron tanks owing to the greater thickness necessary for the requisite strength, and also owing to the provision of heavier flanges for the plate joints. They are more durable than steel tanks ; they are seldom used in waterworks practice

in Bengal, although some examples may be seen in railway undertakings.

The other three types of tanks are designed on the same principle, viz., the walls are made sufficiently strong to span between the roof or the line of stays, as the case may be, and the floor when subjected to load produced by water pressure.

In case of riveted steel tanks, angle iron or tee stiffeners are provided 3 or 4 ft. vertically apart to which ties are fixed at an inclination of 45° connecting similar stiffeners on the floor. The stays and stiffeners are designed to take up the reactions of that portion of the wall. The actual stress on a tie depends, in addition on the angle of inclination to the plates, on the pressure due to its position and the area supported, and also the stiffness of the tank. With shallow tanks, the ties should be at one-third the depth, corner ties may be also put in to stiffen the horizontal angles of the tanks. The plates of the tank are seldom made less than ¼ inch thick.

In case of pressed steel tanks, the standard plates are usually 3 to 4 ft. square, and the flanges are 3 inches wide with ⅝ inch bolt holes. The joints are made up with strip lead $2\frac{3}{4}'' \times \frac{1}{8}''$ inserted between flanges before they are bolted up and caulked tight from the inside of the tank, or with patent jointing material applied in the same way. The lugs are malleable iron castings bolted to the tanks and to the stay bars. The stays are made in standard lengths from flat bars $2\frac{1}{2}'' \times \frac{3}{8}''$ and are interchangeable. Plates can be painted or galvanised as desired. These tanks can be made to any size and to a depth up to 12 ft. The walls of rectangular reinforced concrete reservoirs are frequently designed to span vertically between the roof and the floor, whether they do as a simple slab or as beams between which the slabs span horizontally. In calculating bending moment of the beam or the slab, the ends are considered free and not fixed, as considerable doubt exists as to the degree of fixity that can be afforded. The maximum bending moment of beam under a load uniformly varying from zero at one end to a maximum on the other occurs at a point

0·5771 from the no load end of the beam, and has a value of $\frac{wl^2}{15.5}$ where w= the maximum load at the end, and l= length of beam.

Reinforced concrete rectangular tanks are seldom used in this province, but a notable exception is that at Asansol.

Service reservoirs are usually provided with the following assessories—

1. Inlet and outlet pipes fitted with bell mouths.
2. Overflow and wash-out pipes usually combined with proper valve control.
3. Ladder and Manhole for cleaning and inspecton.
4. A water-level indicator, or preferably a recorder, fixed to indicate water level in the tank.
5. Proper arrangement of ventilation and lightning protection.

The inlet and outlet pipes of both the clear water and service reservoirs are to be so arranged as to keep the water in the best possible circulation, as stassis invariably deteriorate the quality of water which loses its aerated character, becomes flat and insipid, and collects impurities from the air. A difficulty arising out of this kind of stagnation was experienced by Mr. S. C. Chakervurtty, Executive Engineer, Calcutta Water Works, in 1926 of which he writes as follows:—"Owing to certain works being carried out in connection with the suction pipe of two electric pumps, the connection between* chambers 2 and 3 at Tallah had to be cut off leaving chambers 1 and 2 isolated from the pumps so that water was partially stagnating there, I say partially as some water from No. 1 was coming back into chamber No. 4 through the inlet pipes, when the level of water in the latter was low. The bacteriological quality of the water came down from *Negative* in 10 c.c.s to *Positive* in 0·1 c.c. Vigorous steps were taken to find out the source of pollution, as it was then believed to be the case, while

* The Tallah clear water reservoir is an underground reservoir divided into four compartments.

temporary arrangements were made to sterilize the water with electrolytic chlorine. It struck me that motion and light being against germ life, stagnation and darkness must be favourable to their multiplication, and I suggested that the supposed case of contamination was really one of stagnation. It was however not before separate samples from the four chambers were analysed for successive days that the analyst and the Health Officer of the Corporation could be convinced that the case was really one of stagnation and not contamination, and everything became normal as soon as the work in connection with the suction pipe was finished and interconnection between chambers Nos. 2 and 3 was restored."

The following are approximately the costs of different types of tanks per 1000 gallons capacity, when they are about 30 to 40 feet high.

Steel tank on masonry base	Rs.	375
Steel tank on steel staging	Rs.	350
Reinforced concrete tank on masonry base	Rs.	400
Pressed steel tank on steel staging ...	Rs.	300

CHAPTER X

DISTRIBUTION OF WATER.

General Principles—The water for a public supply, from whatever sources it may be derived, is usually delivered in Bengal into one or more reservoirs suitably located within the area to be served. Such reservoirs are generally designed to hold at least one-third the average daily supply, so that when the demand at any time in the day is more than the average supply, the water can be drawn from the storage. These reservoirs are also necessary as a safeguard against accident or temporary breakdown of pumps or the rising main. The water from the reservoirs is conveyed to the consumers by a system of pipes laid in the different roads within the area to be supplied.

The cost of distributing pipes is generally the heaviest item being about 60 per cent. of the expenditure in a water-works project, and their design therefore deserves careful consideration.

The pipes must be economical in size and at the same time fulfil the requirements of an efficient supply. The principal requirements of a satisfactory supply are as follows:—

(*i*) The total quantity should be distributed to different parts of the town in proportion to the requirements of the people living in those areas.

(*ii*) The supply must be maintained at sufficient pressure to meet the objects for which it is used.

(*iii*) The lay out of mains should be such that the water can be delivered to the consumers with a minimum chance of deterioration, contamination or stoppage.

The supply may be given continuously without stoppage, or the water may be turned on only for a few hours in the morning and evening. The former is known as *constant service* and the latter *intermittent service*.

In an intermittent service, the water is turned on only for a few hours in the day and shut off throughout the night. This necessitates the provision of a vat or cistern at every house. The cistern has to be filled when the water is turned on, and the water thus stored used during the hours of the day when the supply is shut off. The consumer has to satisfy his requirements with the quantity available in the cistern which again is liable to depletion by exceptional demand or to contamination. Besides this, water stored in cisterns or vats under the most favourable condition is liable to physical deterioration or pathogenic infection. Moreover, the emptying of mains during the periods that the water is shut off tends to create a vacuum and thus favours the infiltration of subsoil or surface water in the mains from outside through defective joints, thereby causing a contamination of the supply. Another peculiar way in which the contamination of a supply takes place is brought out by a committee appointed by a known corporation in 1929 and is explained in the following portion of its report:— "The liability of leaky water pipes to act as land drains to receive foul matters as well as the land drainage through their leaks is not to be overlooked. Such leaky pipes running full of water with considerable velocity are liable to receive by lateral insuction at their points of leakage external matters that may be dangerous. The latter fact is not recognised so generally as it should be, and ignorance of it has probably baffled many inquiries in cases where water services have, in truth, been the means of spreading disease."

Another disadvantage in an intermittent service is that in the case of a fire during the period of stoppage, no water is available to extinguish it. The chief advantage in this system is that repairs and connections to the mains can be made during hours the pipes are without any pressure. And also that leakage, through defective joints of the supply pipes, distribution and service pipes, is limited to the hours of supply.

In a constant service the water is available to the consumers during the whole twenty-four hours and can be drawn direct from the tap when required without the least chance of

pollution. The whole system of pipe lines remains continuously under pressure, consequently there is the less danger of contamination from infiltration but more from insuction through defective joints.

In the constant service there is a considerable waste through leaky mains, but in the intermittent system, probably a larger amount of water is wasted owing to the running out of unused water in vats becoming dirty or polluted. In Europe and America, public convenience and the progress of sanitary precaution have gradually made it obligatory on local bodies to give a constant supply.

Systems of lay out—There are two systems of distribution usually adopted in India, the *dead end* and *gridiron systems*. The simplest, and perhaps the cheapest, way of arranging distribution pipes is to take the mains and branches independently to the areas to be served by the shortest route ; such arrangements, however, tend to result in a number of dead ends where the supply can be drawn only from one direction and for want of means of circulation the unused water remains stagnant in the pipes. Besides, there is the disadvantage of the risk of stoppage of supply when the supply pipe breaks down in any section. When such a system is adopted, the ends of the mains and branches should be provided with scour valves for periodically washing out the pipes. With the gridiron system, where the mains and branches are arranged so that they form a network, water can always be supplied from two directions, and a partial supply can be maintained even when the mains are under repairs. It is, however, believed that a combination of the two foregoing systems is best suited to the conditions prevailing in India, as most of the towns in this country are generally composed of isolated *bustees* with a limited number of roads linking them. The aim of the designer should be to maintain a partial supply in case of breakdown of a pipe in an important area by linking together the important mains by pipes of not less than 3 to 4 inches diameter. In this way, stagnation and consequent deterioration of water can be avoided, but in this system provision for a larger number of

valves for management, repairs, and detection of waste are necessary.

For the proper alignment of the distributing pipes in a town, it is desirable to start with a large scale up to date plan of the town shewing the roads, public buildings, different bustees or congested areas, and also giving levels representing the surface configuration of the town. Then, the population that will be served by different pipe lines is to be ascertained and marked on the plan. The number of population in different sections of the town can be obtained from the census register and the number of holdings in different streets from the Demand Register of the Municipality ; the average number of inmates per house can be worked out therefrom. Having thus determined the average number of people per house, it is a comparatively easy matter to distribute the population in different roads or pipe lines according to the number of holdings on them. Proper allowance, however, must be made for the increase of population due to the number of births over that of deaths and also for the expansion of trade and industry.

On the plan the proposed mains and branches are marked on the roads, choosing as a general rule the main thoroughfares and those roads which pass through the densely populated parts of the town. In India, as the majority of the consumers draw water from the standposts, it is only fair that provision should be made for water mains and standposts within a reasonable distance of every bustee either existing or likely to be built in near future. There is no definite rule as to the limit of this *reasonable distance* ; it depends upon the local conditions but it is a good practice to provide, if possible, for water pipes within a distance of 200 feet to 300 feet from the property line so as to bring water within an easy reach of the bustees, and also to make the service connection less costly. The number of standposts in the scheme should not only be well distributed but also be proportioned to the number of population likely to be served by them.

In an irregularly built town with limited number of roads, a great deal of experience and judgment is required to lay out

even approximately the most economical system of pipes for distribution. This experience can only be gained by working out the sizes and cost of different arrangements and finding out which is the cheapest. Having thus approximately fixed the position of the mains and branches on the plan, the engineer should inspect the different roads and sections of the town to see if the different parts of the town are efficiently served by the system proposed, and also to see if the lengths of the mains and branches cannot be reduced by making different arrangements. In many cases, it may be found that by some alteration in the alignments of pipes or by taking some main through an unimportant road, a more efficient and cheaper system of pipes for distribution can be worked out.

Having thus determined the lay out of the distribution mains and branches, the second step is to adjust the sizes of all the distributing pipes so that the delivery may take place without undue loss of head by friction in pipes and consequent increase in the cost of pumping. The size of pipes must also be such that the requirements of the different areas of supply are satisfactorily fulfilled.

Principles of Calculating Size of Mains—When proportioning the mains, the following points should be carefully considered :—

(i) VARIATION OF DEMAND—It is essential to estimate the maximum supply of water which may have to be given in a period, so that the pipes must be capable of delivering them without lowering the terminal pressure. This question has been dealt with in pages 20-21 to which a reference should be made. According to Tudsbury and Brightmore, the pipes in England are designed to deliver three times the average daily supply for the whole year. In America, where the rate of average supply is much greater than in England, the maximum demand according to many authors is 200 to 250 per cent of the yearly average. In India, where most of the works are designed on the basis of intermittent supply, the distributing pipes should have larger delivering capacity. The practice in Bengal has

been to make the pipes of such sizes that they deliver at the rate of 1·5 gallons per head per hour in very small towns, 2·5 gallons in medium sized towns and 3 or more in larger towns according to circumstances.

(ii) RESIDUAL HEAD—In Europe and America, the head at the end of any distributing pipe is seldom made less than 60 ft. as water for extinction of fire is usually drawn from these pipes. But, provision for water for fire in a *mofussil* municipality in India makes the water supply schemes so expensive that it goes beyond the limits of the financial resources of local bodies to carry them out. The practice in Bengal has been, therefore, to limit this head to 15 or 20 ft. The residual head must be at least 10 ft. higher than the highest tap at the end of the main which it supplies.

(iii) AGE OF PIPES—The discharging capacity of the pipes is gradually reduced with age, owing to the tuberculation of the inner surface of the pipes where it is uncoated or defectively coated. These tubercles gradually increase in size and spread, and thereby reduce the sectional area of the mains and their discharging capacity. As most of the waterworks in this country are carried out by loan, it is essential that the pipes should be sufficiently large to meet the demand at least up to the end of the period of the repayment of the loan. According to Mr. E. B. Weston, the following are the approximate discharging capacities at the end of the periods mentioned against them.

TABLE 47

Ages of pipes in years	10	15	20	25	30	35
Capacity ...	1	·89	·80	·75	·67	·64

(iv) VELOCITY OF PIPES—The velocity of flow in distribution pipes should not exceed 5 feet, and 2·5 ft. to 3 ft. is

generally considered to be most suitable for pipes up to 18 inches diameter.

Numerous formulæ have been devised by different engineers at different times to estimate the flow of water through pipes subject to different conditions. The literature on the subject is vast, and the results so far obtained are not uniform. The space in this book does not allow this subject to be dealt with in detail. The formulae generally accepted by engineers in this country and elsewhere are the following in calculating the velocity of flow :—

(1) *Kutter's formula*

$$V=\frac{a+\frac{L}{n}+\frac{m}{S}}{1+\left(a+\frac{m}{S}\right)\frac{n}{R}}$$

Where, a=a constant=41·66,

L= ,, = 1·811

m= ,, = 0·00281

n=a variable depending upon the roughness of the interior of the pipe.

R=hydraulic mean radius or depth

$$=\frac{A'}{P}=\frac{\text{area of pipe}}{\text{wetted perimeter}}$$

$$=\frac{d}{4}\text{ for circular pipes when running full.}$$

l=length of pipe in feet.

S=the sine of the angle of slope of the hydraulic gradient.

$$=\frac{h}{l}=\frac{\text{head of water above terminal head}}{\text{length of pipe in feet.}}$$

In calculating sizes of pipes for distribution system, the value of n corresponding to pipes in fair condition is taken to be ·013.

(2) *Santo Crimp and Bruges formula—*

$$V=124\sqrt[3]{R^2}.\sqrt{S}$$

(3) *Tudsbury and Brightmore's formula for 1′ to 4′ dia.*

Rusted pipes $h=\dfrac{lQ^2}{900d^5}$

Where Q=delivery in cft. per sec.
d=diameter of pipe in feet.
h=head lost in friction in feet.
l=length of pipe in feet.

For clean iron pipes $h=\dfrac{lQ^2}{1850d^5}$

(4) The best well known formula and generally considered very suitable and reliable in America is that known as *Hazen and Williams formula*

$$V=a.c.\ R^n\ S^m$$

where a=a fixed constant and equal to $(0{\cdot}001)^{-0{\cdot}04}$
c= a variable dependent on material ; $n=0{\cdot}63$; $m=0{\cdot}54$.

The following values of c are recommended by the authors :—

TABLE 48

Nature of Pipes.	Value of C.
New Cast Iron pipes, straight and smooth ...	140
Ordinary new Cast Iron pipes	130
Old Cast Iron pipes for calculations of future capacity	100
New riveted Steel Pipes	110
Steel pipes under future conditions	95

The calculation of discharges of pipes under various heads or pressures is a laborious task requiring considerable time. Several hydraulic diagrams and tables giving the discharges and velocities of different sizes of pipes have been published as means of saving labour. Such books can be obtained for each of the formula mentioned above. The logarithmic charts IV—VIII give the discharges and velocities of flow for various pipes acording to Kutter's formula, when n=0·013, 0·014 and 0·015 under different heads.

It has been found by experiments by Messrs. E. B. and G. M. Taylor that for uncoated pipes with usual proportions of bends and undulations that the co-efficient of roughness n (in Kutter's formula) was as nearly as ·013. The diagram for ·013 will be used in explaining the method of finding sizes of pipes in the following pages.

Before proceeding further, it may be useful to understand the laws of variation in regard to the head, the diameter and flow in pipes. The following laws are approximately applicable in the majority of formulæ in use, *viz.* :—

(*i*) When the head and length of the pipe, or in other words, the hydraulic gradient remains constant, the delivery varies as the square root of the fifth power of the bore.

(*ii*) When the diameter and length of the main remain the same, the discharge varies as the square root of the head.

(*iii*) When the diameter and head remain constant, the supply will be inversely as the square root of the length.

The table 49 gives the relative discharging capacities of pipes of various diameters. When laid at an uniform hydraulic gradient, the total of the discharging capacities of different branches added together must not be more than that of the main from which they bifurcate. From this table it will appear that for a double discharge, a pipe of double sectional area is not necessary. Thus, the discharge of a 4 inches diameter pipe is not equal to 4 times that of pipe of 2 inches diameter but 5·6 times ; a slight increase in the diameter of a pipe gives therefore a very great proportional increase in discharge.

TABLE 49

Relative Discharging Capacities of Pipes.

Diameter in inches.	Relative Discharging Powers $= d^{2\cdot5}$.	Diameter in inches.	Relative Discharging Powers $= d^{2\cdot5}$.
¼	0·000062	10	0·6339
⅜	0·000172	11	0·8043
½	0·000354	12	1·0000
⅝	0·000620	13	1·2210
¾	0·000975	14	1·4710
⅞	0·001435	15	1·7470
1	0·002003	16	2·0520
1¼	0·003498	17	2·3860
1½	0·005525	18	2·7560
1¾	0·008127	19	3·1530
2	0·011390	20	3·5880
2½	0·019809	21	4·0510
3	0·0312	22	4·5490
3½	0·0459	23	5·0810
4	0·0641	24	5·6570
4½	0·0827	25	6·2620
5	0·1120	26	6·9040
5½	0·1428	27	7·5940
6	0·1768	28	8·3140
7	0·2598	29	9·0730
8	0·3629	30	9·8590
9	0·4871		

The usual calculation that is required in a water supply scheme is to find out the size of a pipe that will maintain the minimum *residual head* at the outlet, when delivering the maximum quantity of water the main is required to supply. The actual difference of level between the water in a reservoir (generally the draw off level and not the top water level) and a tap at the end of the main which it supplies is called *initial* or *static* head. The head lost in friction between the flowing water and the walls of the pipe as denoted by "h" in the formula for the flow of water in pipes is called *frictional head* and the difference between the two is called *residual head*

which causes discharge at the end of the pipe. Small additional losses due to bends and valves are often neglected. In more accurate calculations, the following table giving approximately the length of straight pipe of the same diameter that would cause the same friction as in the fittings may be useful:—

TABLE 50

	Approximate length in feet for Straight Pipe.
Sluice valve (full open)	1·5 feet.
Hydrant with junction pipe	35 ,,
Reflux valve	12 ,,
Bends (with radius=3 to 5 times diameters) ...	3 ,,
Round Elbows	10 ,,
Sharp Elbows and Tees	20 ,,
Foot valve and strainer	45 ,,

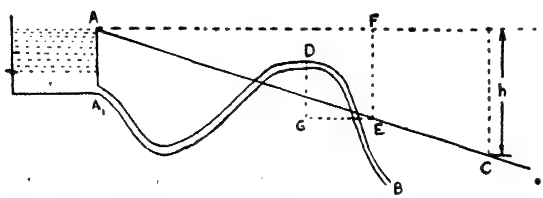


Fig. 94—Diagram showing Hydraulic Gradient.

The hydraulic gradient in case of distribution pipes is an imaginary curve, whose ordinate as measured from the main gives the residual head at any point in the pipe line. Suppose Fig. 94 represents the section of the ground with water main of uniform bore from an elevated reservoir to any point of supply.

If the line of pipe AB be of the length L and if h be frictional head, and if we make AC=L and Sine $Q=\frac{h}{L}$, the pressure at all points of the imaginary pipe AC will be atmospheric, and AC is the hydraulic gradient. If we make AE=AD measured along the pipe line, then EF is the head

lost at D, and this is more than the fall in the pipe line at D. The pressure head at D is therefore negative by the amount that the point D lies above the hydraulic gradient, viz. DG. This is important in practice, because if a partial vacuum exists at any point, air will collect there and will obstruct the flow and even stop it altogether, and should the pipe line rise more than the barometric head above the hydraulic gradient, no flow is possible even theoretically.

The difficulty may be overcome by one of the following ways :—

(*a*) By lowering the portion of the trench for the pipe line through the ridge. This may be found impracticable or costly in some cases.

(*b*) By altering the size of the pipe and thus altering the hydraulic gradient.

The second method is usually adopted in practice. The hydraulic gradient is so altered as to maintain the minimum residual head at the brow of every ridge in addition.

In flat countries, when the calculations show that the residual head is sufficient at the end, the pipeline is rarely above the hydraulic gradient line. But in the case, where a line of pipe first runs over a flat ground and then down a steep slope, it is then also important to see that the hydraulic gradient is well above the main. In practice, the mean hydraulic gradient line is taken to be the line joining the tops of residual heads at these salient points. The distribution system of the portion of a town is shewn in Fig. 95 and a statement giving the method of calculating the sizes of pipes is given in page 251. The plan is prepared in the manner described in page 240 and gives particulars of length of pipes, the levels of the ground or roads, as the case may be, and the population to be served in different portions of the system, etc. In the statement columns 1, 3 and 11 are filled up from the particulars in the plan. The column 3 gives the total population served in each section of the mains. The total population in the area is 21,500, which is shewn against the section of pipe a—b

through which the whole supply is delivered. At b the main b—d takes off and also the supply is reduced by the quantity delivered to 500 persons living between a—b, therefore 21,000 people will be served between b—c as shewn in column 3 against this section. In this way, population served by other sections of the system are worked out as shewn in column 3. The figures in column 4 are worked on the basis of maximum draw off, fixed in the manner described previously; in this case,

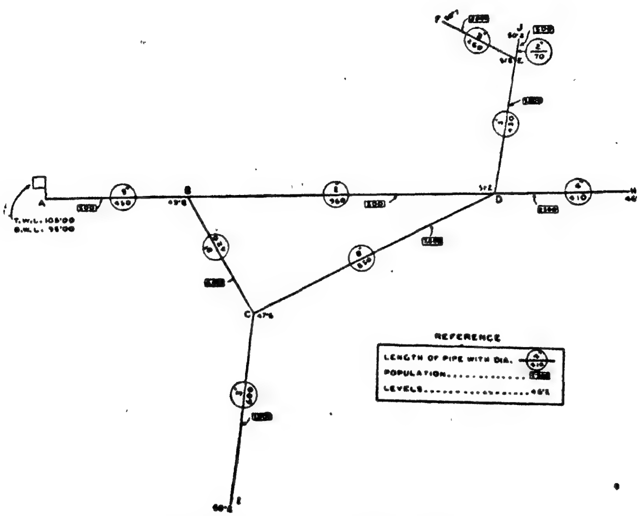


Fig. 95—Distribution System.

it is assumed to be 1·5 gallons per head per hour. For working the figures in column 5 of mean hydraulic gradient, it is first necessary to find out by which route the trunk main is to be laid, whether along a b c d e f, or along the direct route a b d h. The determination of such route requires experience and judgment. The best way of judging the route is to calculate the sizes of the pipes by both the routes and find out which works cheaper. In this case, it is evident that the route of the trunk main must be along a b c d e f. The average hydraulic gradient will be equal to the head available between these points, i.e., 95·0(D.W.L.) – [50·7(G.L.) + 15(Residual Head)] divided by

the length of the pipe line a b c d c f, *i.e.*, 2410 which is equal to 1·21 per 100 ; in this way, the average hydraulic gradients for the other portions of the pipe are found out. Next, an inspection of the diagram for the flow in pipes will give the sizes of the pipes that will deliver the required quantities of water in column 4 under the heads almost equal to, or somewhat less than, the heads given in column 5. The actual loss of head due to friction per 100 ft. length of pipe to deliver the quantities of water in column 4 is taken from the diagram, and the total losses are obtained by multiplying these figures by the corresponding lengths of the pipe line. The level of the terminal head is obtained by deducting the total loss of head from the level of the initial head, and the residual head is obtained by subtracting the corresponding ground level from this. The average hydraulic gradient is the curve joining the residual heads at different points of the pipe line.

Compound Main—It has been so far assumed in this chapter that the problem is to compute the size of a pipe of given length to deliver a required quantity of water. But it is frequently necessary to find the delivery of an existing pipe line. In such cases, two problems generally arise.

1. The discharge from a compound main of varying diameter.
2. The discharge from "looping" pipes or gridiron system of pipe lines.

In both cases, an algebraic expression can be given to denote the relation between discharge and frictional head, but in practice, the following method is generally found convenient.

(1) Some discharge is assumed, and then the total head lost in friction in different sections of the mains when delivering such an amount of water is computed. Then, divide the square root of the actual available head by the calculated head, and multiply it by the assumed discharge to obtain the actual discharge. It is explained in the following example.

TABLE 51

Statement of Calculations for Determining Sizes of Pipes.

Reference by letters on plan.	Length of Pipe in ft.	Population to be served.	Maximum draw off in gallons per hour.	Average hydraulic gradient fall in 100'-0".	Diameter of pipe in inches.	Level of initial head.	Loss of head of pipe in ft. n=·013 Per 100 ft.	Loss of head of pipe in ft. n=·013 Total	Level of terminal head.	Terminal ground level.	Residual head in ft.	Remarks.
1	2	3	4	5	6	7	8	9	10	11	12	13
A—B	450	21,500	32,250	1·22	9"	95·00	0·92	4·14	90·86	49·80	41·06	A, B, C, D, E, F is assumed to be main.
B—C	420	20,500	30,750	1·28	9"	90·86	0·84	3·53	87·33	47·60	39·73	BD branches off at B.
C—D	850	15,500	23,250	1·39	8"	87·33	0·94	8·00	79·33	51·20	28·13	CI ,, ,, ,, C.
D—E	430	5,500	8,250	1·90	5"	79·33	1·60	6·88	72·45	51·80	20·65	DH ,, ,, ,, D.
E—F	260	2,000	3,000	2·60	3"	72·45	3·60	9·36	63·09	50·70	12·39	EJ ,, ,, ,, E.
B—D	960	500	750	2·24	2"	90·86	2·25	21·60	69·26	51·20	18·06	Head on the main pipe at D=79·33.
C—I	600	1,000	1,500	3·70	3"	87·33	0·96	5·76	81·57	50·20	31·37	
D—H	410	2,500	3,750	4·42	4"	79·33	1·67	6·85	72·48	46·20	26·28	
E—J	70	500	750	10·03	2"	72·45	2·25	1·65	70·80	50·20	20·60	

Reservoir D.W.L.—95·00
Maximum demand 1·5 gallons per head per hou

It is required to find the discharge of compound main consisting of 2000 ft. of 12 inches pipe, 1500 ft. of 8 inches and 1200 ft. of 6 inches dia. pipes for total loss of head = 30 ft. minor losses of head are neglected.

Assume a discharge as stated before, and find out from the diagram the head that will be necessary to give this discharge from the diagram. In this case, we have assumed the discharge to be 30,000 gallons an hour.

TABLE 52

Dia. of pipes.	Loss of head to delivery 50 galls. per minute.	Total loss in the section.	Equivalent length of 6″ pipe to give same loss of head and delivery.
12″	·14	2·8	$2000 \times \frac{\cdot 14}{7} = 40$
8″	1·47	22·0	$1500 \times \frac{1\cdot 47}{7} = 315$
6″	7	84	1200
	Total ...	108·8	1555·

Now, there is 30 ft. loss of head in 1555 ft. of 6″ pipe or 1·94 per 100 for which the diagram shows for 6″ pipe discharge = 15500 gallons an hour.

In the first way, this discharge

$$= \frac{30{,}000 \times \sqrt{30}}{108\cdot 8} \text{ gallons}$$

$$= 15{,}100 \text{ gallons.}$$

(2) For the second case, take the following example:—

12″ dia. A | 1200 ft. of 10″ dia. pipe. | B

2500 ft. of 5″ dia. pipe.

C

The total quantity delivered by the 12″ pipe at A is 25000 gallons per hour. The loss of head between A and B is required to be found out, and also the delivery of AB and ACB.

It is evident that the loss of head in the pipe AB must be the same as that in the pipe ACB, otherwise there will be two pressures at B, which is impossible, and also the total of the discharge of AB and ACB must be equal to 25,000.

The simplest method will be to assume a loss of head between AB. Take it to be 30 ft. Then, along AB there is a loss of head of 2·5 ft. per hundred, and along ACB 1·2 ft. per hundred, and deliveries under those heads are 72000 and 6000 gallons an hour. Then, the delivery for 10 inches pipe= $\frac{25000 \times 72000}{78000}$ =23,000, and for 5 inches pipe=2000, and loss of head along AB=0·25 per 100, and that along ACB=0·12 per 100. These are approximate figures and are all that are required for practical purposes.

Frequently, an engineer has to consider increasing the capacity of existing mains on account of increase in the population of any particular area in a town. In many cases, this is done by putting in a larger size of pipe, or by putting in a boosting pump, such pumps being, as a rule, electrically driven, where electric current is available.

CHAPTER XI

DATA REGARDING PIPES AND ACCESSORIES.

Water mains in this country generally are of cast iron or steel ; recently reinforced concrete pipes have also come into use. The latter is still in an experimental stage and has not been generally accepted.

Cast Iron Pipes—Cast iron is acknowledged to be the standard material for distribution pipes in towns and cities all over the world. The durability of cast iron pipes under conditions prevailing in municipal water-works is not exceeded by any other materials. Cast iron pipes laid 250 years before in France, and more than a century ago in England, are said to be still in use. In India, we have pipes 50 or 60 years old which are still in service, and in very good condition. These pipes are generally coated with Dr. Angus Smith's coating to prevent corrosion. This coating is composed of the following ingredients :—

Best crude asphalt or bitumen ..	44	per cent.
Coal tar freed from water naphtha ..	55	,,
Resin	0·1	,,

The coating was first used in Manchester Waterworks in 1849. In applying this coating, the pipes are heated to a temperature of 600°F and then immersed in the bath of solution (the temperature of which is maintained at 350° to 400° F.) for sufficient length of time to enable them to acquire the full temperature of the bath, when they receive a tough adhesive coating of bitumen or pitch. When they are exposed in a warm atmosphere, the outsides of the pipes are sanded to render them less liable to run out.

Cast iron pipes are generally made of best grey pig iron remelted in a cupola. In modern practice, these pipes are

almost universally vertically cast with a head of 1 ft., which is afterwards cut off in a lathe. This process has the advantage of giving a more uniform thickness to the pipe wall than when they are cast horizontally, under which conditions the metal is liable to float the core. In vertically cast pipes, the metal is of more uniform quality, and porous places are avoided by cutting off the top portion of the casting, where bad metal or scum is collected. Cast iron pipes of less than 3″ dia. are generally manufactured in 6 ft. lengths, and occasionally in some foundries, in 9 ft. length.

The thickness of a water pipe is largely governed by the working conditions, practical casting considerations and the condition and depth of the soil in which it is laid. Different foundries had until recently different empirical formulæ for the thickness of these pipes. These dimensions have now been standardised and the British Standard Specification for cast iron pipes gives details of thickness etc., under various conditions of working, and manufacturers have now generally adopted this specification in their foundry practice. The particulars of dimensions, thickness, weights etc. of these pipes are given in Appendix IV.

Steel Pipes:—Steel pipes are of three types, riveted, welded and solid drawn depending on the bore of the pipe and the purpose for which they are used. These pipes have been in use in some places of Europe and America for over 35 years. Steel pipes are generally used for long rising mains, aqueducts, especially when they are of large diameter 3 or 4 feet, or more, and on bridges or other structures where strength with least weight is required, and also in places where the cost of transport is heavy.

In London, out of a total length of 3376 miles of water mains, about 1071 miles of pipes are of steel, while the soil conditions, it is reported, offer a severe test of the underground pipes. Sir George Handover, the Chairman of the sub-committee appointed for investigating the relative bursting efficiency of cast iron and steel mains, summarised the reliability of steel pipes with the words "We are of opinion·

that in the future steel pipes adequately protected should with due regard to cost be used for trunk and high pressure mains in order to obviate as far as possible the occurrence of innundations due to fracture". Considering the effects of age on the carrying capacity of pipes, steel pipes after 30 year's use deliver 20 per cent. more water than cast iron pipes of the same bore.

Previous to the introduction of welded and solid-drawn pipes, steel pipes were generally made of a single plate, whose edges were riveted together or by winding a long narrow plate spirally and riveting the spiral lap. The same practice is followed to the present day in the manufacture of riveted pipes. They are generally made in sections 15 to 30 ft. in length. The jointing of these pipes are generally carried out by flanges at the end of the pipe formed by riveting angle iron rings on to the pipes. Owing to the difficulty of riveting, these pipes are not generally manufactured in diameters less than 9 inches. These mains, however, generally range in sizes from 2 to 7 feet internal diameter and for any high pressure. Recently, very large sized mains have been manufactured and laid in connection with the Calcutta and Bombay Waterworks.

The strength of a riveted pipe is calculated in the similar way as the shell of a boiler, and a similar allowance is made for the riveted joints.

Bends and specials are made of two types, one is known as *lobster backed bend,* and the other *smooth bend.* Fig. 96 shows the types of bends ; the first one has been adopted in Calcutta Waterworks.

The riveted steel pipe is generally assumed to have a friction of 15 to 20% greater than that of good cast iron pipes. Owing to the process of manufacture, there is considerable practical difficulty in applying a thoroughly efficient coating for protection, and consequently the life of these pipes is much shorter than other classes of steel pipes.

Welded Pipes.—This class of pipe has made considerable encroachment upon the field hitherto occupied by cast iron

pipes. These can be made of thinner material, and at the same time of equal strength and have the same carrying capacity.

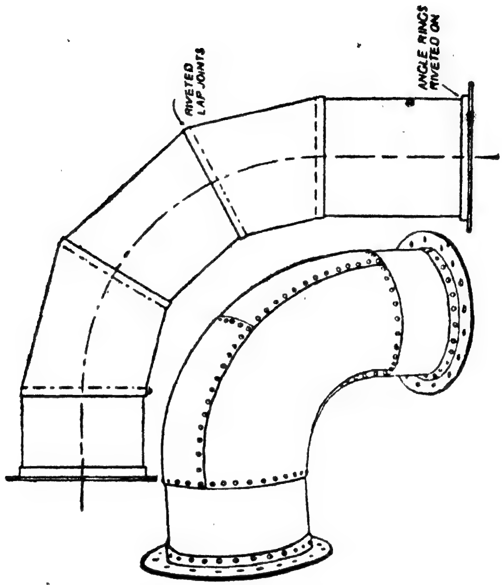


Fig. 96—Lobster Bend and Smooth Bend.

Welded pipes are made by carefully bending steel plates accurately cut to proper width into the form of the pipe, and then welding the longitudinal seam by the oxyhydrogen flame or otherwise, and pressing together when under weld heat with rollers at the sides of the pipes. Welded pipes can be manufactured in all sizes and in lengths up to 40 ft. depending on the diameter and the largest size of plate available. This class of pipe is generally 15 to 20 per cent. lighter than riveted pipes.

Solid Drawn Pipes—This class of pipe is drawn from suitably shaped solid billets of mild steel or high tensile steel to the required size and thickness. The pipes are hot rolled to bring them to the required dimension by means of suitable rollers.

This process gives exceedingly fine finish, great flexibility and strength, but the cost of production is much greater than welded and riveted pipes. The solid drawn pipes are manufactured up to 12 inch bore and in lengths of about 18 to 35 ft.

Steel pipes are generally made from Siemens-Martins' open hearth steel having a tensile strength of 24 to 48 tons per sq. inch of section. These pipes from 1 inch diameter and above are generally coated with same composition, which is used for protecting cast iron pipes and wrapped outside after coating with jute hessian soaked in the Angus Smith solution. Up to 9 inches diameter, however, they can be had with galvanised coating as well to prevent corrosion. The galvanised pipes are commonly used for service connection.

Advantages and Disadvantages of Steel Pipe—Steel pipes have the following advantages over cast iron pipes:—

(*i*) Lightness—consequently cost of transport and handling is less.

(*ii*) Greater strength.

(*iii*) Being made in lengths 18 to 35 ft. or longer, there are less number of joints and less leakage.

(*iv*) Being flexible, can be bent to any radius without altering the shape or size of the joint.

But they are more liable to corrosion, and the thickness being small in comparison to cast iron pipes, special ferrules are necessary for house-connections; whereas in the case of cast iron pipes, a hole can be tapped at any point and an ordinary ferrule inserted. Bends, branches and other specials in a steel main of more than 12 inches diameter are generally made of cast iron; smaller sizes can be had of steel.

The following conclusions were drawn by Mr. W. Hutton in a paper read before the Engineering Conference at Simla from the use of Steel Pipes in Waterworks for over 20 years.

(1) That experience of their use shows variable results.

(2) That Steel Pipes are cheaper than Cast Iron Pipes, but the superior lasting quality of the latter is beyond doubt.

(3) That as the life of steel pipes apparently almost entirely depends on the effectiveness of the original coating, and on the subsequent laying of the steel pipe in the trench with its coating undamaged or restored, the greatest precautions are necessary to ensure good results.

(4) That it is preferable to use steel pipes, which have been dipped in a bath of Angus Smith composition, wrapped with hessian cloth and redipped in the bath instead of steel pipes, which have only been dipped and not subsequently wrapped with hessian cloth.

(5) That steel pipes should preferably be used for delivery or rising mains, and that the distribution pipes in towns should preferably be of cast iron pipes to avoid excessive corrosion due to the presence of nitrates in the made earth of streets.

(6) That as the corrosion or pitting of steel plates takes place to a less extent in dry or permanently wet locations, such places are suitable for the laying of steel pipes.

(7) That before adopting steel main in preference to a cast iron main, it is necessary to determine chemically the nature of the soil, in which the steel pipes will be laid, in order to determine whether or not such soil contains deleterious salts likely to cause corrosion in the steel pipes. Corrosion of steel is generally attributed to the following causes:—

(1) It is due to the pipe being laid in ground subject to alternate conditions of dryness and wetness, thus causing the presence of air and Carbonic Acid gas around the steel pipes.

(2) It is due to the presence of pin holes in the protective coating, which admit air and moisture to the surface of the steel plate from which pitting rapidly develops.

(3) It is due to the presence of deleterious salt in the soils in which the pipes are laid.

(4) Corrosion is due to the impurities in the steel plates from which the pipes are manufactured, as experience shows that whereas one pipe may be pitted after a short time in the ground, the adjacent pipes in similar soil are not affected.

The coating of steel pipe is a subject to which a great deal of attention has been devoted lately, as a result of which several compositions of coating have been brought out to meet the different conditions of the soil.

Pipe Joints—There are many forms of joints used in practice to connect up different lengths of pipes, and these may be classified as follows:—

(*i*) Socket and Spigot. (Fig. 98).
(*ii*) Screwed and Socketed. (Fig. 100).
(*iii*) Flanged. (Fig. 101 & 102).
(*iv*) Welded. (Fig. 103).
(*v*) Riveted.

SOCKET AND SPIGOT JOINTS—This form of joint is used both for cast iron and steel pipes. They offer considerable advantages over other types of joints. Figs. 97 to 99 shows typical joints used for cast iron and steel main in practice. The long sleeve joints shown in Fig. 98 for steel pipes are very suitable for places, where there is danger of subsidence due to bad ground or heavy traffic.

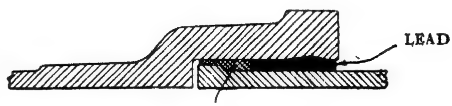


Fig. 97—Socket and Spigot Joint.

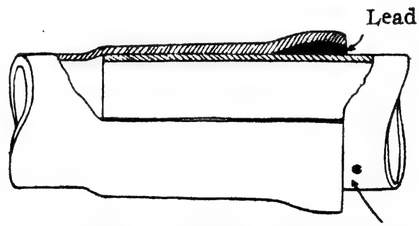


Fig. 98—Long Sleeve Joint.

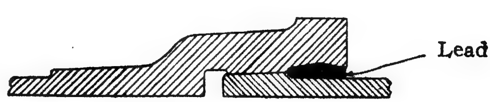


Fig. 99—Turned and Bored Joint.

Another form of joint (Fig. 99), known as *turned and bored joint*, has been used for cast iron pipes in many places, where economy is the principal consideration. This form of joint has been quite satisfactory and has stood the test of time and efficiency, both in this and other

countries in the West. This joint is formed by the adhesion of the turned surface of the spigot end of the pipe against the bored interior of the socket end of another pipe. The only jointing material used in this class of pipe is soap solution for slipping in the turned end of the pipe.

The socket and spigot joints are made by caulking in several turns of yarn of tarred hemp, and afterwards filling up the remaining annular space with cast lead or lead wool, and finally caulking the lead or lead wool until the joints are thoroughly tight. The lead is generally poured in a molten state into the joint by placing a thick strap of stiff clay, and when it becomes cold, the clay strap is removed and the lead properly caulked with caulking tools. The lead should be soft pig and melted in a ladle of sufficient capacity to fill at least a socket at one operation. Care should be taken to ensure dryness of the socket, otherwise the molten lead will be blown out into the jointers' face.

As a substitute for cast lead, lead wool for this class of joint is now coming into use. The manufacturers claim that the disadvantage of cast lead joints attendant upon pouring molten lead into joints not entirely free from moisture is obviated. This is especially the case, when repairing joints of pipes laid below ground and under water pressure. It is claimed that joints with lead wool offer greater resistance to vibration and are more efficient, even when made under water. The lead wool, as the name implies, consists of ropes of loosely coiled fibrous threads of lead ; these are caulked into the socket turn by turn until the joint is completed. The caulking should be done by special curved caulking tools to fit the annular space, and carried out with a hammer not less than 4 lbs. in weight. The efficiency of such joints largely depends on the jointer, and the cost is somewhat more than cast lead joints. Lead wool joints can be adopted to all sizes of pipes, but cast lead joints above 36″ diameter pipes are undesirable. The tools required for jointing pipes with lead are given in Fig. 122.

The quantity of cast lead, lead wool and yarn required for different sizes of pipes are given in Appendix IV.

Lead joints allow considerable latitude in the change of direction of the pipe to be laid without the use of bends, and can be adopted to uneven ground. This class of joints is easily repairable, as the defects can be remedied by a few stroke of caulking tools. Water mains with lead joints are more flexible, and less subjected to troubles attendant upon the expansion of pipes, as each joint is more or less an expansion joint.

The main with turned and bored joints, on the other hand, are less flexible and the joints at frequent intervals required to be run with lead, when the main deviates slightly from a straight alignment. The turned and bored joints are however cheaper than cast lead joints, and require less skilled labour. Pipes with turned and bored joints can be laid much quickly even in an wet trench, as practically nothing is required to be done after the turned end of the pipe is driven home into the socket in the trench. As a precaution against leakage, we should think, however, the space inside the socket should be leaded. In case of pipes less than 4″ diameter, turned and bored joints are not at all satisfactory. They should always be lead jointed pipes.

Screwed and socketed joints—This form is almost universally used for steel or galvanized iron pipes of small diameter. The ends of the pipes are screwed outside, and the sockets inside with gas threads. The sockets are usually screwed on to the end of the pipe of each length of pipe as tightly as possible, the threads being first besmeared with a paste of white lead or graphite. At intervals, in this system of jointing, long screws of sufficient length to allow a backnut and a socket to be completely screwed are provided for replacement or repairs to pipe. Fig. 100 shows a long screw joint with backnut at the back. Obviously, with this arrangement running joints are not required, and long lengths of pipes can be lowered into the trenches after jointing.

Flange Joints (Fig. 101):—This form of joints is usually adopted in places where the pressure is high, and where the temperature changes are great.

The British Engineering Standard Committee went into the question of proper proportions of flanges for different pipes under different working conditions in 1890, and prepared a

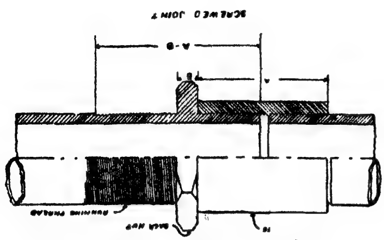


Fig. 100—Long Screw Joint.

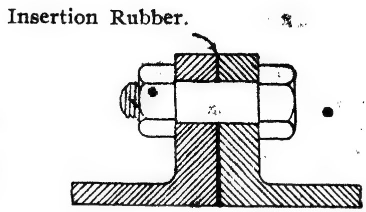


Fig. 101—Flange Joint.

complete set of table giving full particulars of flanges, their thicknesses, diameters and weights etc. A list based on these particulars will be found in Appendix VI.

The flanges for cast iron pipes are cast integrally with the pipe, while flanges for steel pipes can be fixed in any of the following ways, viz., expanded on, screwed on, riveted on or welded on. Fig. 102 shows typical joints of each of these.

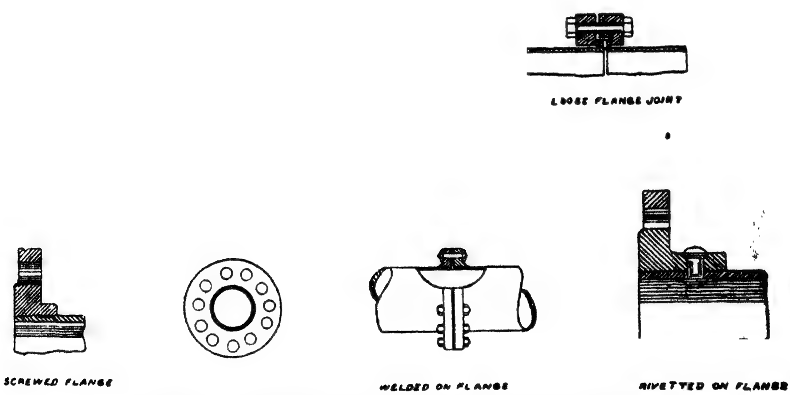


Fig. 102—Different kinds of Steel Pipe Joints.

Expanded or loose flange joints are suitable for pressures up to 300 lbs. per square inch. Flanges should be faced across the joint, and the bolt holes should be drilled and cutter-barred or faced at the back, so that the nuts may bed down satisfactorily. Flanged joints are made water-tight by means of some

jointing materials between the flanges, as shewn in the Fig. 97. The most satisfactory jointing material for water is a thin piece of rubber sheeting known as *insertion rubber.* Some times, canvas smeared with red lead on either side is also used. The bolts are tightened uniformly, and the tightness of the joint is tested with a *feeler.*

Flange jointed pipes are very rarely used in the distribution system. They are suitable for suction and delivery pipes of the pump. This form of joint is also used as fitting up tail pieces of sluice valves, air valves and other water main accessories.

WELDED JOINT (Fig. 103) :—A development in the design of joints of steel mains has recently been made consisting of welding a long sleeve socket and spigot joint, either by oxy-acetylene or oxyhydrogen blow pipe. An 18 inches gas main has been laid with this kind joint at New Castle N.W.S. with considerable success.

Fig. 103—Welded Joint.

RIVETED JOINT—This form of joint is generally restricted to riveted steel pipes. In practice, steel pipes of several sections are usually riveted together at the shop enough to make a length of 20 to 40 feet according to the size of the pipe. Larger pipes are riveted with lap or butt joints, every other section of the lap joint pipe being made smaller than the others and fitting tightly into their ends or each end is slightly tapering, one end forming the inside and the other outside of a lap joint. The diameter of the rivets are generally made twice the thickness of the plate up to a maximum of $1\frac{1}{8}$ inches. The spacing should be such that will not only make the joint amply strong, but will make it water-tight under all conditions of working. The pitch of the rivets seldom exceeds 3.5 times the diameter of the rivets. Riveted joints, whether made in the shop or in the field, should be thoroughly caulked and

tested under water pressure. The mains for the Calcutta water supply extension scheme now under construction is exclusively of riveted steel pipes with lap or butt joints.

Requisites of Distribution System—A water works distribution system generally consists of pipes, specials, valves, hydrants, meters and other appertenances necessary for conveying, measuring and controlling the water required for the consumers, with provision for extension for future requirements. In the previous section, particulars of pipes have been dealt with, and a general description of some of the other accessories are given below.

SPECIALS—Pipes should be laid in straight lines both horizontally and vertically as far as possible. Where this is not possible, special castings having the same bore are used to connect up the lengths by means of a curve. These special lengths are designated as 1/32, 1/16, 1/8 and 1/4, which represent that the angle of deviation from straight on one side of the bend is $11{\cdot}25^\circ$, $22{\cdot}5^\circ$, 45° and 90° respectively (Fig. 104). Lead jointed socket and spigot pipes can, however, be laid in slight curve, but their permissible curvature is limited by the depth of their sockets and thickness of the joints. The minimum opening of the joint must never be less than ¼ inch. When socket and spigot pipes are laid in a curve, they must be laid to proper radius without any kink.

Branches from trunk mains are connected up either by a *Tee, Y* or a *Cross* piece, according to circumstances. The specials used in connection (Fig. 104) with a water distribution system, together with their weights and dimensions, are given in Appendix IV.

AIR VALVES—It is important in designing a distribution system to provide air valves of sufficient capacity at changes of gradients for the purpose of releasing the large quantities of air in a water main when it is being charged, and for the escape of air, which afterwards accumulates under pressure. Such valves are also necessary, as stated before, to admit air to the pipe line to prevent formation of vacuum in case of a rapid discharge of water following a burst at some lower point of the

pipe line. This is specially important in case of large steel mains, which are usually thin and cannot withstand any external pressure without collapsing

Air valves are generally of two kinds, namely single acting and double acting, the latter having two outlet chambers with balls, one of which lets out large volume of air when the main is being filled, and the other acting under and allowing the escape of air which accumulates afterwards. These are shewn in Fig. 105.

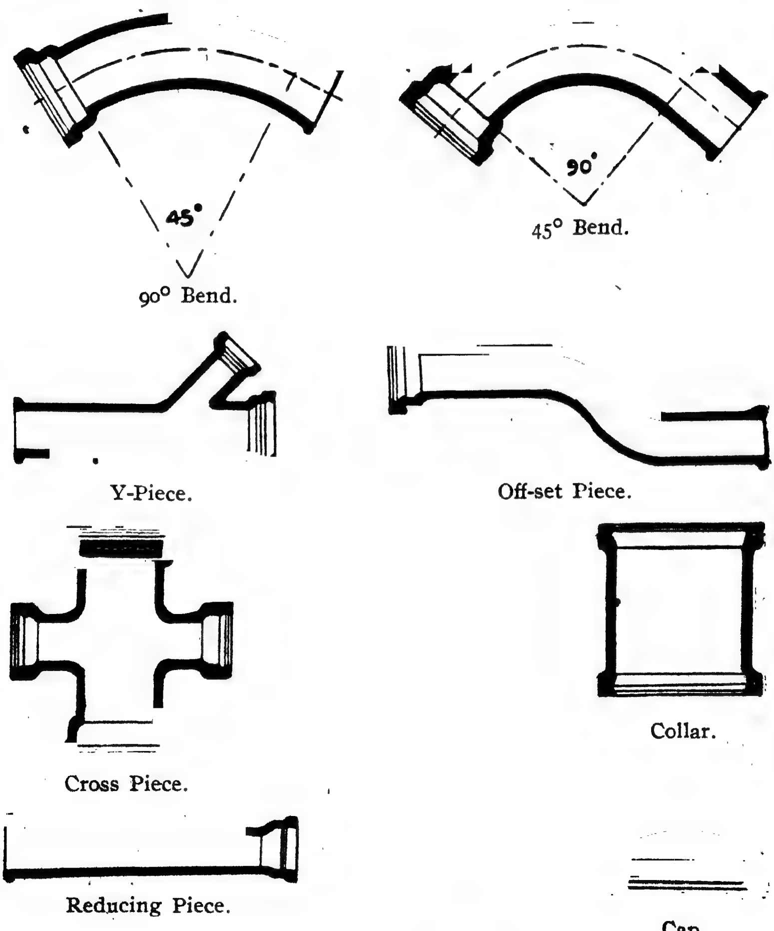


Fig. 104—Specials.

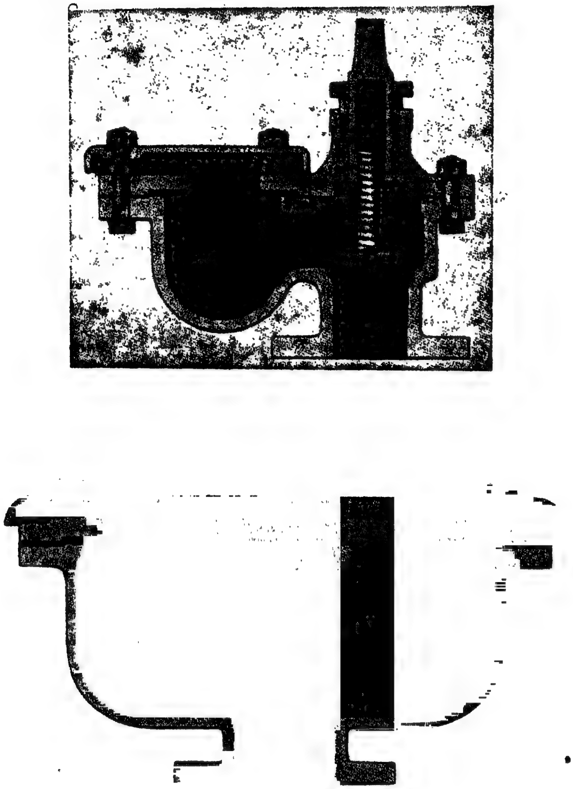


Fig. 105—Air Valve.

The operation of these valves is simple ; when there is no water in the pipe, the ball drops down, but as soon as water reaches the ball during charging, it quickly begins to rise owing to its buoyancy and the velocity of the escaping water. With the rise of the ball from its seat, the air accumulated inside the pipe starts escaping and continues to do so until the ball finally closes the air outlet of the valve. The ball must be of sufficient weight to overcome the upward pressure, acting on the orifice on which the ball rests, due to the pressure of air or water in the main.

The sizes of the valves usually adopted are as follows:—

TABLE 53.

Size of mains.	Sizes of air valves.
Up to 4" dia.	2"
Between 5" to 8" dia.	3"
Between 9" to 18" dia.	4"
Between 19" to 30" dia.	6"

The valve should be fixed inside a masonry chamber to prevent surface water draining into main when empty, and also to prevent fouling. The ball is generally made of India rubber or vulcanite according to pressure. In a double acting air valve, a stop valve is generally provided in the centre for the examination or repairing of the ball without emptying the main. Single acting air valve can also be had with gun metal cock at the base for shutting off the mains for repairs. The valve seat and the spindle should be of gun metal.

The purpose of air valve is often to a certain extent served by standposts, when they are properly located.

WASHOUT VALVES—Similar precaution is necessary against the deposits of dirt collected in the lower reaches of pipes laid across a valley or depression. This is effected by the provision of a scour or washout valve which is ordinary sluice valve, fitted on a branch, off the main at the lowest point of the depression. The valve when opened discharges into a low land or surface drain alongside the pipe. These valves need be only about one-third the size of the main pipe. The washout valve chamber is shewn in Fig. 106. In America, special type of branches are used for this purpose as shewn in Fig. 107.

SLUICE VALVES—(Fig. 108). Mains are sub-divided and controlled by means of sluice valves at suitable points. Sufficient valves should be used, so that the breakage of a main will cause inconvenience to the smallest number of consumers.

Valves should be fixed on the hydrant or standpost branches for the same reason. Proper distribution of sluice valves often considerably reduces the waste of water through breakage of

Fig. 107—Washout Branch.

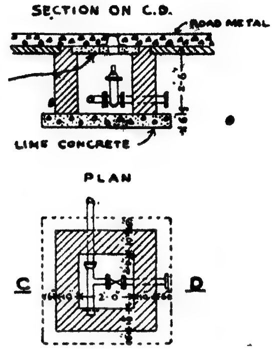


Fig. 106—Washout Valve Chamber.

mains. The faces of the valve should be of gun-metal—two in body and two in wedge-shaped door. All these faces must

Fig. 108—Sluice Valve.

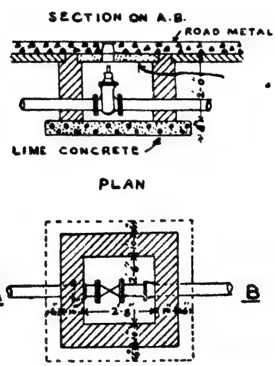


Fig. 109—Sluice Valve Chamber.

be accurately machined and hand scraped to correspond to the recesses in body and door respectively. The spindle and the

nut should be of forged gun-metal ground all over with machine-cut square thread. The gland and the stuffing-box should be fitted with gun-metal bush. It is better to have an indicating mark on the wheel to show the direction in which the valve opens, as the absence of such indication often leads to error and accident. Sluice valves are generally housed in chambers for easy access when opening or closing. A typical design of these chambers is shewn in Fig. 109.

In Appendix IV, standard dimensions of sluice valves and flanges are given for reference.

STANDPOSTS—Standposts are very often of varied design; the type shewn in Fig. 110 has been used in many places in Bengal, and has been found to be fairly satisfactory. The

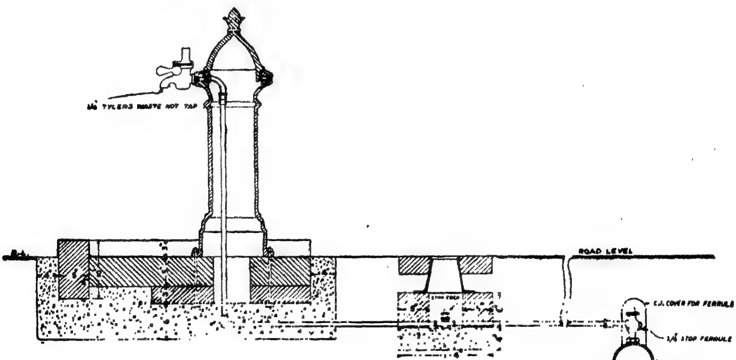


Fig. 110—Stand Post and connection with Street Main.

number of standposts should be proportionate to the number of users, so that no person has to wait for more than a minute or so to obtain a *Kalsiful* of water. One important point is to be remembered in this connection, viz., the tap shall be self-closing and adapted to the minimum working pressure. This is extremely necessary to prevent unnecessary waste due to carelessly leaving the tap open.

CISTERN FOUNTAINS—These have been sometimes found convenient, especially in case of small supplies. These are

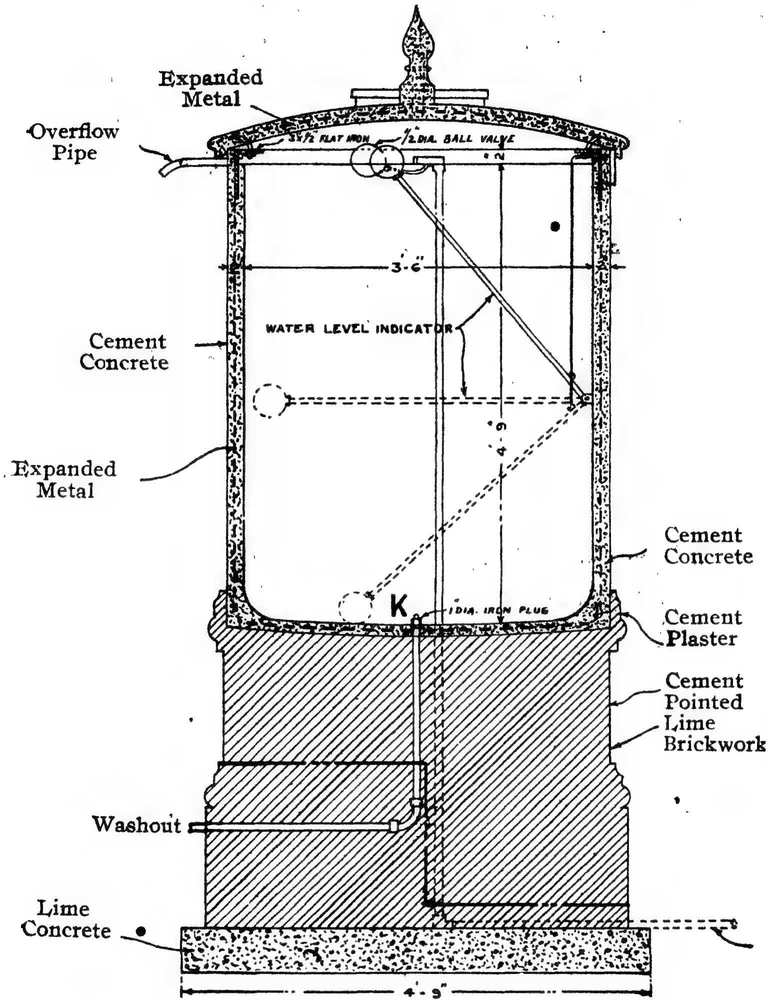


Fig. 111—Cistern Stand Post.

simplest in construction and consist of galvanized iron tanks of about 100 gallons capacity placed three or four feet above ground. The cisterns are filled at times of least demand and controlled by a ball valve. The waste-not taps are fixed on the sides of the tank for drawing off water. These are very suitable for places, where the pressure is low, particularly during the hours

of maximum supply, as they can be filled during the time the supply is cut off and act as balancing tanks when the demand is maximum. They are, however, liable to contamination when not properly looked after. Recently, re-inforced concrete tanks of this type have been built for Asansol Waterworks. The capacity of these tanks is 400 gallons each. (Fig. 111), and the cost is Rs. 180.

REFLUX VALVES—These are some times used at the foot of a long slope, up which a water main is laid, to prevent running back of water in case of accident to the pipes in the lower reaches. It is also used on the rising main close to a pumping station to prevent draining back of the water pumped. A common type of reflux valve is shewn in Fig. 112 ; the body and

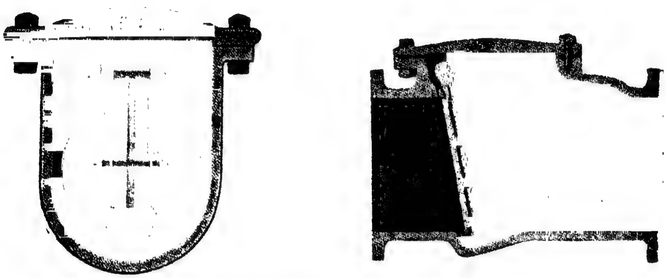


Fig. 112—Reflux Valve.

flap are of cast iron, the seating and face are of gun metal. They offer great resistance to the flow of water and are seldom used in a distribution system except in a hilly district. To overcome the resistances offered by this design of valve, the Tilting Disc Reflux Valve has been put on the market, and is the outcome of experiments carried out by Glenfield & Kennedy, Ltd. An illustration of this valve is given in Fig. 113.

In essentials, the device consists of a circular disc, with bevelled edge, hinged about a fixed pivot offset from the plane of the seat. This permits of the door of the valve offering a streamline shape to the flowing water and so reducing losses through the valve. On account of its pivoted movement, the closing of the valve is free from slamming, which is a valuable

point in overcoming tendencies to water hammer and the stresses, which are occasioned by the sudden closing of the ordinary type of reflux valve.

Fig. 113—Tilting Disc Reflux Valve.

RELIEF VALVES—These are used in pumping mains, or in mains laid in a hilly district, where there is a considerable different of level in various parts of the main with a view to prevent the internal pressure exceeding a certain limit in the lower parts. These valves can be adjusted to any pressure desired.

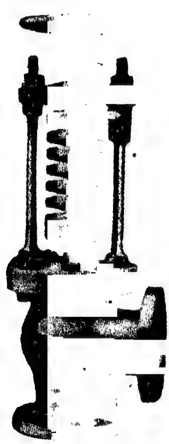


Fig. 114—Relief Valve.

The Valve shewn in Fig. 114 operates in a way similar to the safety valve in a boiler, and can be adjusted to suitable pressure by the adjustment of the spring.

BREAK PRESSURE TANKS—In certain hilly districts, these are extremely useful for keeping the pressure in the parts within the safe limit of the pipe. Fig. 115 shows the section of a break-pressure tank commonly used. The water discharges into the tank through a ball valve, which is closed by the float when certain level is reached. The head on the outlet pipe is thus brought to the head of water in the tank. The apparatus has a bye-pass arrangement, and also a washout and overflow from the tank. Occasionally Pressure Reducing Valves as

shewn in Fig. 116 may be used for this purpose. This type of valve is arranged, so that it will maintain a fairly constant

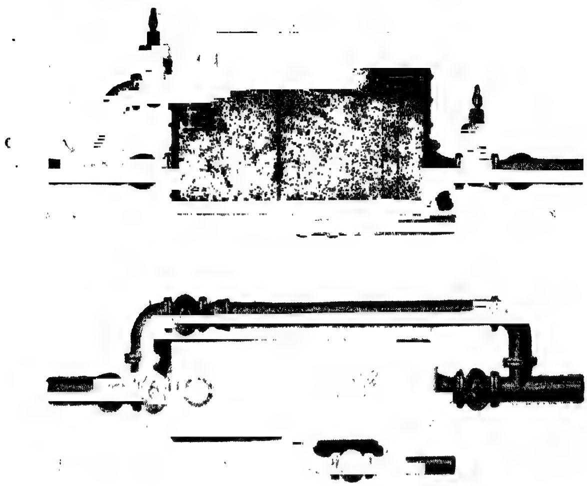


Fig. 115—Break Pressure Tank.

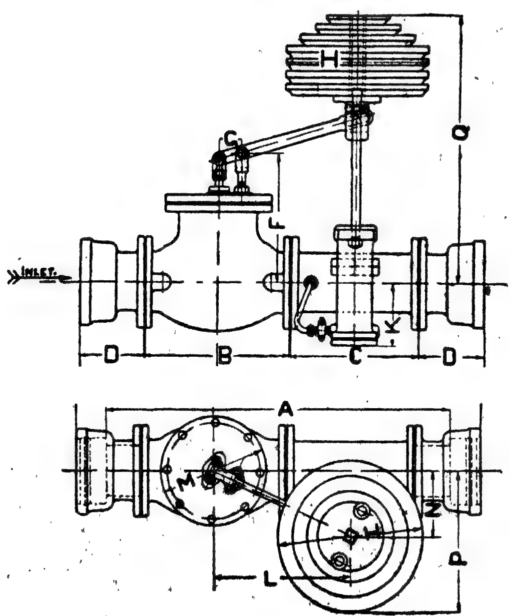


Fig. 116—Pressure Reducing Valve.

pressure on the discharge side of the valve, no matter what the fluctuation of the pressure may be, and should the pressure on the inlet side of the valve rise above that for which it is set, the valve closes and re-opens again when the pressure falls. It is capable of a wide variation of setting.

Expansion Joints—In case of lead jointed pipes, the expansion and contraction are fairly provided for by the flexibility of joints. In case of pipes with flanged, riveted or welded joints, provision for expansion and contraction is necessary—especially when long lengths of pipe are exposed to the atmosphere. A typical expansion joint of sliding type is shewn in Fig. 117. This form of joint is meant for straight pipes.

Fig. 117—Expansion Joint.

The sliding portion and the gland are fitted with a gunmetal casing to resist corrosion. The joint is provided with anchor bolts which, while they allow the joint to move within certain limit, prevent any danger of the joint pulling out. This form of joint is suitable for pipes up to 18 inches diameter.

Ball and Socket Joints—When pipes are laid across a stream, morass or undulating ground, this form of joint is very useful as they follow the profile of the surface of the trench by bending without racture. Besides, these joints can be made on the shore, and the whole line can be laid like a cable under water. The majority of these joints are made with bearing surfaces of lead on iron. All-iron ball and socket joints have been used with gaskets. These joints allow deflections of 6° to 17° on

an average of 10°. Flexible jointed Cast iron pipes are generally made in lengths of 9 to 12 ft. Fig. 118 shows a common design of this form of joint.

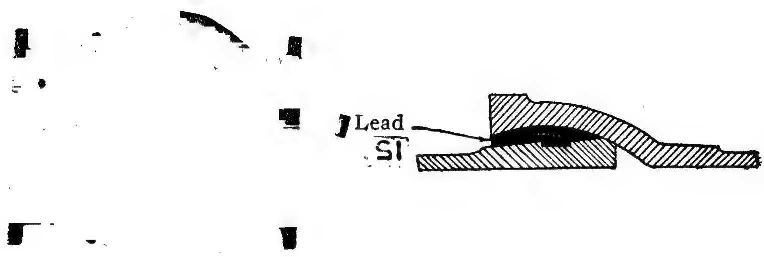


Fig. 118—Ball and Socket Joint.

Hatch Boxes (Fig. 119). These are useful for cleaning pipes and recoating them with suitable non-corrosive paint and should be inserted at intervals of 500 to 600 feet. It is generally

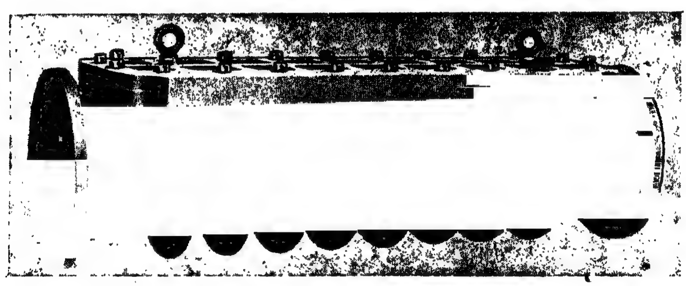


Fig. 119—Hatch Box.

acknowledged that, both cast iron and steel pipes conveying water are more or less susceptible to corrosion unless effiectively protected with some suitable paint. Corrosion manifests itself on the internal surface of the pipes in the form of tubercles, which vary in thickness and size according to the character of water conveyed and with age. The thickness in some cases has been found to be nearly of an inch. The incrustation of pipes

reduces considerably their carrying capacities and decreases the efficiency of the distribution system.

Scrapers and brushes of various types and makes are available in the market, and have been successfully used to remove corrosion forming inside the pipes without taking them out from site. Scrapers and brushes are introduced inside the distribution system through hatch boxes, and are pulled either by water pressure or by means of a winch. A type of these, as manufactured by Messrs. Glenefield & Kennedy, is shewn in Figs. 120 and 121.

These have been successfully employed in many places in this as well as in other countries.

Fig. 120—Pipe Scraper.

Fig. 121—Pipe Cleaning Brush.

But, simple removal of tubercles from pipes are not of much use, as corrosion takes place with greater rapidity than before, and the delivery capacity is brought to its original level in much shorter time. To obviate this difficulty, it is necessary to re-coat the internal surface of the pipe with suitable non-

corrosive material. The following process was employed successfully in Bloemfontein by Mr. W. R. White, City Engineer.

"This is carried out by means of specially designed spraying apparatus which is inserted into the main to be re-coated and, after adjustment, is drawn through the main by means of a winch. The spray nozzle is of the revolving type and is mounted on the end of a container, which contains sufficient bituminous material to re-coat the length of main under treatment. The necessary air suppiy is conveyed to the spraying apparatus by means of a rubber air line laid through the main, and which travels along with the container during its progress through the pipe.

After scraping and flushing out, lengths of approximately 600 ft. are isolated by means of cuts. To thoroughly dry out the main and ensure the removal of all moisture, an air blast from a large rotary blower or fan is directed into the length of the main under consideration. As a rule, one hour's application of air blast is sufficient to thoroughly dry out the interior surface.

After the spraying operation is completed, the air blast is again turned on to assist the drying of the bitminous material, and if necessary, a second or even third coat can be applied in a comparatively short period. The cuts then made good, and the main is ready to be recommissioned. This procedure is repeated with successive lengths of main until the whole line has been scraped and re-coated.

Two coat work is always advisable, as this ensures that any pin holes or defects, which may occur in the first coat, will be remedied during the application of the second or more coats."

The following table shows the gain in delivery after scraping pipes, and is extracted from Glenefield & Kennedy's pamphlet on the scraping of water mains by Glenefield process.

TABLE 54

Name of place.	Diameter main in inches.	Length of Main.		Cost. £	Percentage of Gain in delivery after scraping.
		miles	yds.		
Clackmannanshire, C.C. ..	6	3	1430	117	80
Cockermouth	6	7	880	470	99
Kirriemuir	15	6	0	87	33·7
Merthyr-Tydvil	12	5	617	318	30
	14	6	770	178	82
Plymouth	24	1	880	300	44
Tarapaca (So. America) ..	5, 7, 9	56	0	714	27.28
Waterford	13	8	0	212	40

Laying and Jointing—This is a very important work in connection with a waterworks, and the manner; in which it is executed, materially affects the running cost when the scheme is in operation.

During the course of preparation of a scheme, it may not be always possible to finally locate the exact line of pipes in reference to the existing bridges, culverts, water-mains, surface drains etc., but this must be done before the work is started, and the details are to be carefully worked out. In the final alignment of the pipe, care should be taken to see that the road surface or permanent structures are least interfered with during repairs, and the pipes are least subjected to the danger of contamination through leaky joints from surface drains or sewage and where any such risk exists, and is unavoidable the water main should be put inside another pipe of larger diameter and the annular space filled in with bitumen or cement grout. After the lines of the pipes have been finally located, the exact centre line of the pipes, the situation of each bend, branch valves or other specials are to be carefully marked on the plan, and the site of each special must be carefully marked on the ground. This is necessary to minimise difficulties and unforeseen obstacles that generally hamper the progress of the work.

Considerable care and judgment are also necessary to deposit pipes and distribute them in such a way that the exact number and length of each class of pipe required for different roads are available without double carting, and that the traffic, pedestrians and house-owners have the minimum cause of complaint. It is, therefore, very desirable to survey the different sections of the town with a view to find suitable places for stacking the pipes ; and careful instructions should be given to intelligent assistants to accompany the cartmen to proper destination. Two inches diameter pipes are more fragile and should be stored in safer place than the others. Valves and specials should be deposited in depots central to the different portions of the work. All pipes and specials before laying should be carefully examined to see if they are cracked or damaged in any way, and also if they are up to the specification. Cracks in pipes cannot always be easily detected by striking the pipe with hammer and are only detected when tested under pressure. In cutting off the cracked pipes, the cut must be made 4″ to 6″ above the crack, because the crack may be lengthened while cutting the pipe.

The water mains in this country are usually laid with a cover of 3 to 4 feet to protect them from fracture under heavy traffic, and also to prevent the water in the pipe being unduly heated. This has been found to be quite satisfactory, at least in Bengal.

The gang for pipe-laying work may be divided in four batches, viz.,

(1) Carrying party.
(2) Excavating party.
(3) Pipe-laying party.
(4) Trench-restoring party.

The first party lays down the pipe alongside but clear of the location, specials being placed in position marked for them on the ground. The second party is meant for the excavation of the trench to the right line and level. The bottom of the trench should be fairly graded, and with this object level pegs should be inserted at intervals. Spoil should be on the one side of the trench, and pipe on the other. The width of the

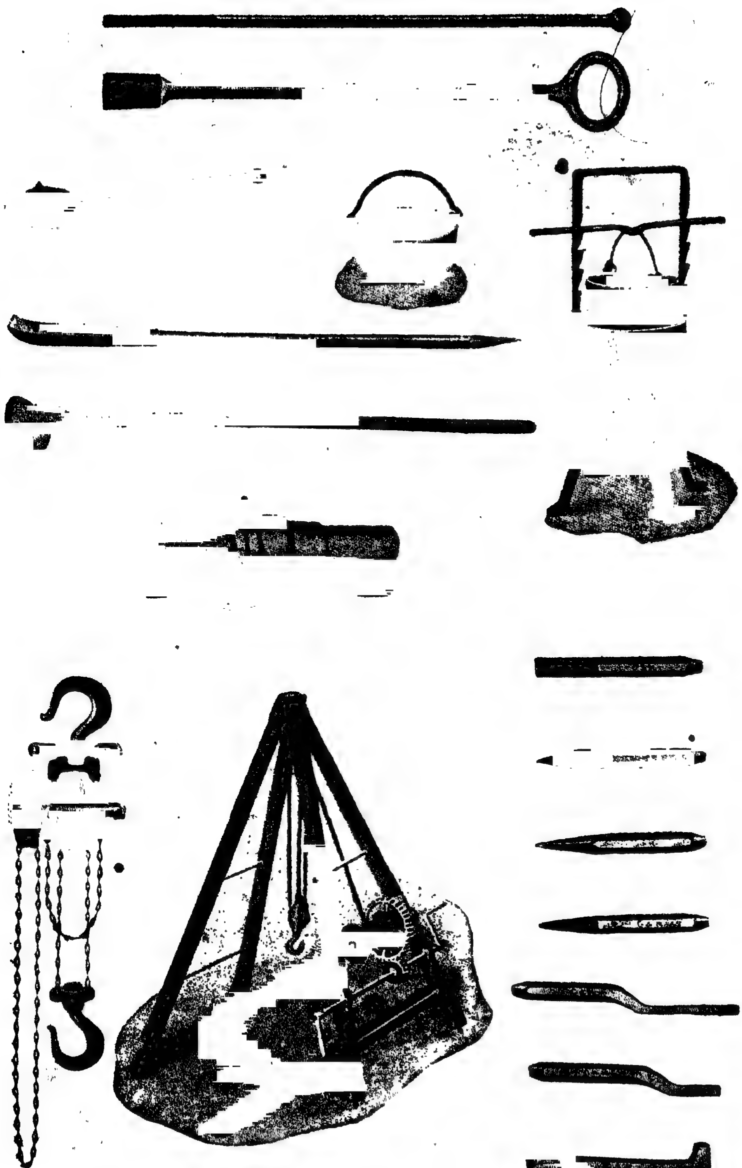


Fig. 122--Pipe laying Tools. Calking Tools.

trench should be at least a foot wider than the outside diameter of the socket of the pipe, and in no case, less than 2′-3″ at the joint which is the minimum width in which a pipe-joiner can work conveniently and satisfactorily. Before the actual pipe laying is started, at least 400 to 500 ft. of trench should be fully excavated, and the pipe-laying gang should be so proportioned that their work proceeds at the same rate per day as the second party.

The third party follows the excavation party. It should consist of one or two plumbing mistries and suitable number of coolies. The inside of the pipes should be first cleaned with a brush tied at the end of a bamboo or long rod, and then the pipes are jointed in the usual way. The tools necessary for lead-jointed pipes are shewn in Fig. 122. The pipes should be laid with their socket ends all pointing in one direction, and it is the usual practice to place the socket end of the pipe in the direction of the flow of water, but this cannot always be arranged, and in the case of mains fed from both the direction, one half of the pipes should be laid in one direction and the other half in the opposite direction, and the ends connected together by means of a collar joint as shewn in Fig. 123. In descending slopes, however, the direction of socket should be reversed, otherwise there will be difficulty in pouring the lead for joint, and air space is likely be left.

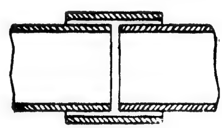


Fig. 123—Collar joint.

The fourth party is intended for filling in the trench when the test is completed. The earth should be well punned in layers not exceeding 6 inches in depth and watered. When the earth is properly consolidated, the original metalling, which is kept separate from ordinary soil during excavation, is well rammed in position over a layer of brick-soling. The whole surface is gradually restored to its original condition. The height of filling should exceed more than 1 inch per foot of depth of trench over its original surface. No trench, however carefully refilled, can be restored to its original surface condition at once. They are always liable to subsidence, so that

whatever surplus earth that is available should be stacked in convenient places to fill up the subsidence. This gang should also be employed in collecting surplus pipes and stores, and carting them to the central store and marking the position of mains and fittings on the ground by suitable permanent marks.

When the day's work is closed, care should be taken to see that the ends of sections of pipes laid have been properly closed with wooden blocks. The workmen must not be allowed to store their tools inside them ; similar precaution should be taken at the commencement of every day's work to see that the ends of the pipes are clean and nothing is lodged inside them.

The cost of laying pipes varies very widely, as is but natural, owing to the various conditions under which the work is executed in various places. The figures given in Appendix V afford a rough guidance for estimates of similar work under careful supervision.

CHAPTER XII.

SERVICE CONNECTIONS

What they consist of—The service connection is the link between the street main and interior piping in the consumers' premises. In Bengal, these connections (Fig. 124) usually consist of a stop or adjustible ferrule fixed on the municipal main, a stop-cock on the communication pipe to enable the supply to be cut off during repairs, a stop-cock box, a meter, a meter-box with locking arrangement, and a length of communication pipe. Some times, owing to the indiscretion of the local bodies, the meter and meter-box are omitted. In service connections, the term *communication pipe* is usually applied to the portion of the pipe, which extends from the distribution pipe in the street to the boundary of the consumer's premises. The distribution pipe is applied to that pipe, which conveys water from the storage reservoir or pump to the street end of the communication pipe.

The service pipes are usually from ⅜ inch to 1½ inches diameter and are made of galvanized iron. The life of these pipes seldom exceed 10 or 15 years, and in America many copper

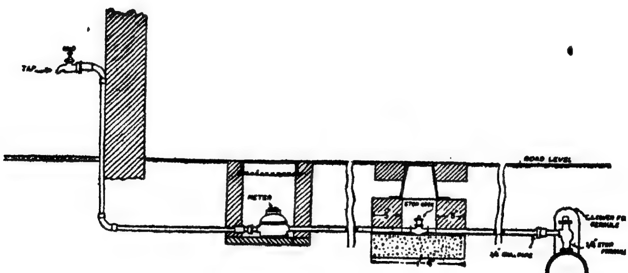


Fig. 124—House Connection with Street Main.

or cement lined pipes are being used in place of galvanized iron pipes. The street main is first drilled and tapped, and

the ferrule screwed in position. As a rule, the smaller mains are tapped on the top, and in case of larger ones, the connection can be made on that side of the main on which the premises of the consumer is docated. The types of ferrule usually used are shewn in Figs. 125 & 126. The stop and bend ferrules are

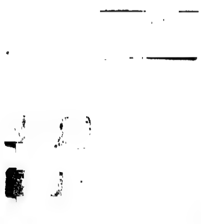


Fig. 125—
Bend Ferrule.

Fig. 126—Stop Ferrule.

intended for top connections, and the straight ferrules for side connections. The size of the connection ferrule depends on the annual rate paid by the consumer. After the communication pipes and the interior system of piping within the consumer's premises together with all fittings are fitted and fixed, the plug valve of the ferrule or the stop-cock is opened, and the water is freely admitted into the house-service. The service pipes should be laid as far as possible at right angles to the street main and along the shortest route to the consumer's premises, and must not pass through another man's property. They should be laid at least to a depth of 2 ft. 6 inches below surface of the ground. No service pipes should be laid so as to pass through any sewer or drain, or through ashpit or manureholes, or through any place where the water conveyed through such pipe is liable to contamination, or may escape undetected when the pipe becomes defective. When the laying of service pipes,

through such objectionable places, cannot be avoided, special precaution must be taken to prevent them from contact by an exterior cast iron pipe or by some other suitable means. When service pipes are encased in another cast iron pipe, the annular space between them should be filled in with cement grout or bittumen.

The pipes, ferrules, stop-cocks and other fittings should be strong, durable and of the best manufacture. All draw-off cocks, stop-cocks, ferrules and other similar fittings should be made of best hard brass or gun metal, and of pattern approved by the municipality. The municipality generally keeps samples of such fittings in office for inspection, and each apparatus bears the name of the manufacturers. Stop-cocks and meters are generally put inside surface boxes (Fig. 127) for protection and for easy access.

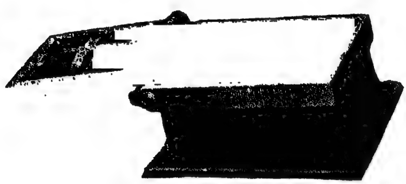


Fig. 127—Surface Box.

How it should be executed—Although consumers have every right to make their own arrangement for the execution of works in connection with their service connection, considering every aspect of the question, it is very desirable however that they entrust this work to the municipality, and thereby get the work done under more efficient and expert supervision. The municipality having more experienced engineer will be more competent to judge the suitability or otherwise of the materials, pipes and fittings supplied. A uniform standard of work and materials will be maintained under this arrangement, and there will be consequently less cost and trouble in the upkeep. The road surfaces will not be unnecessarily broken up, and will be quickly and satisfactorily restored, and the arrangement involves

no responsibility on the consumer. Besides, the work will be done according to a programme, which will be an arrangement in conformity with road-repair scheme Lastly, the municipality will not have to deal with many persons for complaints regarding trench-filling. But so far this arrangement has not found favour any where in Bengal.

Method of connection with Steel Main.—Full particulars of steel and galvanized iron pipes and specials are given in Appendix IV. The service connection from a cast iron street pipe is a comparatively simple affair ; the cast iron pipe generally has shell of substantial thickness, which can be drilled and tapped satisfactorily to make a watertight joint under all working conditions But in case of steel pipes, where the thickness of the shell is very small, considerable difficulty used to be expe-

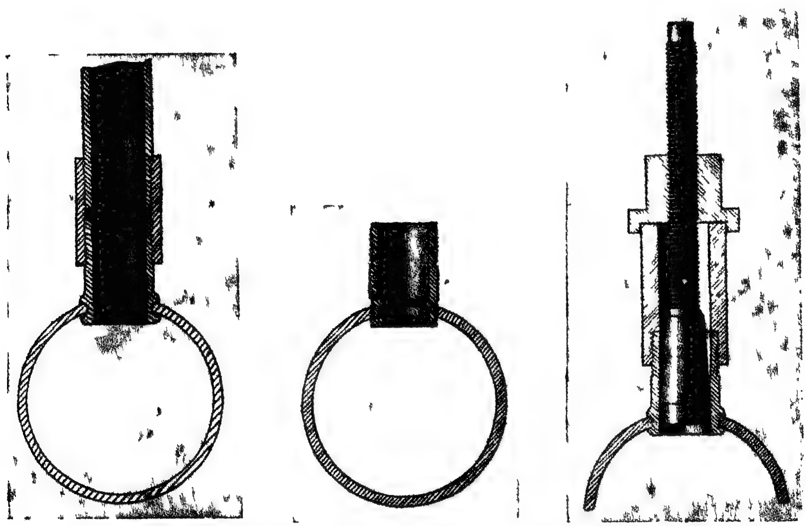


Fig 128—Woodall and Parkinson's Expansion Nipple

rienced in making satisfactory service-connections. But with the introduction of Woodall and Parkinson's expansion nipple (Fig. 128) in the market, the difficulty has been very considerably overcome. In this, the natural strength is utilised by reveting the end of the nipple on the inside of the street main. The

expansion nipple is a length of steel tube screwed at both ends with an internal annular bead at the lower end projecting 1/16 inch inside the tube. The main is first drilled and tapped in the ordinary way. Then, a tapered mandril, having the larger diameter equal to the full bore of the nipple and the other end provided with screwed stem and nut, is inserted into the pipe without the nut. Then, the nipple is put in position and screwed right home into the main so that lower threads disappear. The nut of the mandril with the lengthening sleeve is screwed firmly into the nipple. Then, by means of a ratchet or roller spanner, the nut is turned until the mandril becomes loose, which happens only when the bead end of the nipple has expanded and becomes securely riveted to the main. The saddle joint is then made as shewn in Fig. 129. The gutta

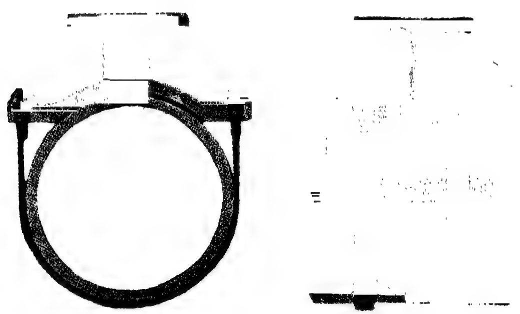


Fig. 129—Cast Iron Saddles.

percha ring fits tightly over the nipple, and the saddle is pressed down upon it by the screwing of a socket. Recently, further improvements have been made in this type of joint ; now an additional bead of similar projection has been added outside the main. The expansion of two beads on two sides of the shell of the pipes forces the thread of the nipple into the tightest contact with the shell of the pipe, and forms a rigid and water-tight connection to any main without any saddle joint. These

joints have been tested under a pressure of 2000 lbs. per sq. inch and found very satisfactory.

Supply to Water Closets—With the introduction of sewerage systems into some of the mofussil towns of Bengal, a fresh demand is being imposed upon the supply from service-connections, and also a new avenue of waste is being opened.

The supply should be given through a cistern fixed on the top of the roof of the consumer's premises. The cistern should have a capacity of at least 60 gallons per each seat. The water is admitted through a ball valve at the top of the cistern, and the supply is drawn off two or three inches above the bottom, while a washout pipe with a stop-cock is fixed at the bottom. The overflow pipe is fixed one inch over the top water level of the cistern, and has a sectional area equal to twice that of supply pipe. A non-return valve should be fixed at the point where the supply pipes to cistern branch off from the communication pipe. The cisterns are usually made of galvanized iron, the life of which is seldom more than 4 or 5 years. Pressed steel tanks are gradually coming into the market ; they are coated with *Angus Smith's* coating and should last longer than galvanized tanks.

There are several types of syphon cistern for flushing latrines. The one shewn in the Fig. 130 has been found to work satisfactorily. The following points are to be observed in selecting a flushing syphon. It should have a capacity of at least 2 gallons per flush. The bearings must be bushed with gun metal to ensure smooth working and easy replacement when worn out. The level of the float must be of brass or other suitable alloy. The ball should be of copper of substantial thickness, and must have sufficient clearance, so that during its rise and fall it must not touch the sides or top of the syphon cistern. No pipe, by which water is supplied to a flushing cistern, should communicate with any

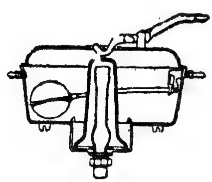


Fig. 130—Flushing Cistern.

other part of the water closet, nor be connected with any distribution pipes inside the premises, as this is sure to lead to waste and the use of deteriorated water from tanks may cause epidemic.

CHAPTER XIII.

WASTE PREVENTION AND METERING.

Nature of Waste—The waste of water in a public water supply is the quantity not drawn and consumed for the legitimate and useful purpose of the consumers. There are very few cities in the world, where the water is not wasted in one way or another. Waste of water develops coincidently with the development of water supplies. Generally, the waste is due to the failure of the consumers to appreciate the value of water as a commodity, and therefore they think that water can be used without regard to economy. The waste may either be intentional, careless or through ignorance. A considerable quantity of water is also wasted on account of defective management. Theoretically, every gallon of water delivered into the distribution system should be available for the use of the consumers. But, this is hardly practicable, as small leaks in the numerous joints of the distribution and service-systems do not appear on the surface, and losses therefrom are practically non-preventable. Mr. H. C. Adams reports that it has been found by experiment with small holes or varying diameter that leakage through them will occur to the extent shewn in the following table under a pressure of 45 lbs. per sq. inch (104 ft. head) in the main.

TABLE 55.

Diameter of hole in inches.	Leakage per hour in gallons.	Leakage per day in gallons.
1/32	15	360
1/16	27	648
1/8	160	3,840
3/16	192	4,608
1/4	450	10,800

In a system of supply, there may be many leaks of larger magnitude in service pipes and other mains, and these leaks may remain undetected for years together. From the above table, it will appear that the quantity of water lost through $\frac{1}{8}$ inch hole under 104 ft. head is sufficient to meet the requirements of a fairly big village in Bengal containing 1000 population.

In this connection, the following extracts from Flinn, Weston & Bogerts' "Waterworks Handbook", with regard to leakage of water from distribution pipes in other countries, will be interesting:—"Geo. T. Deacon, 1894, satisfied himself, by investigation of many water supplies in England and other countries, that more than half of all water pumped is, in the average system, lost by leakage. The same year Sir Fredrick Bramwell showed that in over one hundred British cities and towns provided with meters, $66\frac{2}{3}$ per cent. of the water was thus lost. W. S. Johnson, 1907, showed an average of 52 per cent. of water accounted for in 3 cities with every tap metered, and an average of 48 per cent. accounted for in 21 cities with an average of 89 per cent. of all taps metered. According to State of Board of Health of Massachusetts, 1900, no city of that state having over 90 per cent. of taps metered accounts for over 62 per cent. of water furnished, while one fully metered city finds but 37 per cent. of its supply registered. . . . . Dexter Brackett, 1895, stated that it is not practicable to reduce waste below 15 gallons per capita in large cities, and that this figure can be reached only by universal use of meters, and adoption of thorough methods for detection of underground leaks."

Unfortunately, in this province no attempt has been made either by the municipal authorities, or government administrative department, to reduce the waste on this account or devise means for its prevention. These authorities believe that the service connections are only responsible for the waste of water, and by the effective prevention of this alone, the problem of waste prevention will be solved.

History of Waste Prevention—Formerly, the supply of water in towns in Europe and America was generally inter-

mittent and limited to only a few hours in the day; even as late as 1891, nearly 35 per cent. of the water supply of London was on the intermittent system. Subsequently, as a measure of sanitary precaution and public convenience, a constant supply was attempted in many towns with the result that the demand increased enormously owing to the prolongation of period of waste. Liverpool was provided with a constant supply in 1858, but the rate of consumption increased to such an extent that in 1865 it became necessary to revert to old system of supply. Though this produced temporary relief, the consumption soon increased owing to the unmitigated waste from street mains and service pipes. Even where a constant supply was maintained, a considerable portion of the street mains, service pipes and fittings had to be removed at an immense cost. The excessive use of water in this way not only increases the cost of delivery to the consumers, but also adds to the expenses in different parts of the waterworks plant.

The first attempt to reduce waste was by house to house inspection. This was adopted both in Europe and America, and proved to be fairly successful in determining the number of fixtures inside premises and their condition at the time of visit. It did not, however, improve the situation very much, as a poor fixture could not be converted into a good one at the bidding of an inspector, nor were the consumers likely to be as careful to prevent the water from running to waste as when an inspector was paying his visit.

The next attempt in Great Britain to remove this difficulty was by a careful examination at a depot of the corporation of all the fittings and fixtures intended to be used. Searching tests were applied to them with a view to see that they fully complied in every respects with the prescribed specification, and only those which passed such tests were permitted to be used within the limit of water supply. But when the test, that could be done under the circumstances to prevent leakages from domestic fittings, had been done, the unaccounted-for waste due to other causes was found to be very considerable. The unaccounted-for water is the portion of water flowing into a distribution without

coming to the use of the consumers ; this usually comprises the waste in the distribution and service pipes and their fittings. The following table from "Water Works Hand Book" will give an idea as to the percentage of unaccounted-for water in well metered cities.

TABLE 56.

City.	Per cent. of taps metered.	No. of Years.	Consumption per capita gals.	Per cent. not accounted for.
Brockton, Mass. . . .	83 (100)	7 (5)	34	32 (30)
Boston, Mass. . . .	....	2	91	34
Cleveland, O	49	1	96	21
Englewood, N.J. . .	100	1	...	52
Fall River, Mass. . .	94 (100)	4 (7)	37	22 (13)
Hackensack, N.J. . .	100	5	...	40
Hartford, Conn . . .	99	1	62	39
Lawrence, Mass. . .	86 (92)	3	46	33 (39)
Milwaukee, Wiss . .	79	1	89	16
Ridgefield, N.J. . .	100	3	163	18
Madison, Wis. . . .	92	8	49	37
Syracuse, N.Y. . .	72	1	108	19
Taunton, Mass. . .	42	7	57	32
Ware, Mass.	100	1	44	39
Wellesley, Mass. . .	100	4	52	43
West Orange. N.J. .	100	1	...	20
Woonsocket, R.I. .	87	1	29	24
Worcester, Mass. . .	95 (96)	1 (11)	68	42 (28)
Yonkers, N.Y. . . .	97 (100)	6 (1)	83	45 (17)

The problem of waste-prevention was considerably solved by the invention of the *waste-water meter* (Fig. 139) by G. F. Deacon of Liverpool water suply. Its action is described by the author as follows :—

"When water is passing through a main and supplying nothing but leakage, the flow water is necessarily uniform, and any instrument which graphically represents that flow as a horizontal line conveys to the mind a full conception of the nature of the flow, and if by the position of that line between the

bottom and top of a diagram, the quantity of water is recorded, we have a full statement of not only the rate of flow but of its nature. We know in short that the water is not being usefully employed. In the actual instrument, the paper diagram is mounted upon a drum caused by a clockwork to revolve uniformly, and is ruled with vertical hour lines and horizontal quantity lines representing gallons per hour. Thus, while nothing but leakage occurs, the uniform horizontal line is continued. If, now, a tap is opened in a house connected with the main, the change of flow in the main will be represented by the vertical change of position of the horizontal line, and when the tap is turned off, the pencil will resume its original vertical position, but the paper would have moved like the hands of a clock over the interval during which tap was left open.

. . . . Now all these uses of water, whatever kind they may be, produce some such irregular diagrams as these, which can never be confused with the uniform horizontal line of leakage but are always super-imposed upon it. It is this leakage line that the water works engineer uses to ascertain the truth as to the leakage and to assist him in its suppression. As an example of one mode of applying the system, suppose that a night inspector begins work at 11-30 P.M. in a certain district of 2,000 persons, the meter of which records at the same time a uniform flow of 2,000 gallons an hour, showing the not uncommon rate of leakage of 24 gallons per head per day. The inspector proceeds along the footpath from house to house, and outside each house he closes the stop cock, recording opposite the number of each house and the exact time of each such operation. Having arrived at the end of the district, he retraces his steps, reopens the whole of the stop-cocks, removes the meter diagram, takes it to the night inspection office and enters in the "night inspection book" the records he has made. The next morning the diagram and the night inspection book are in the hands of the day inspector, who compares them. He finds, for example, from the diagram that the initial leakage of 2,000 gallons an hour has in the course of 4½ hours night inspection has fallen to 400 gallons an hour and

that the 1,600 gallons an hour is accounted for by fifteen distinct drops of different amount and at different times. Each of these drops is located by the time and place records in the book and the time records on the diagram as belonging to a particular service pipe, so that out of possibly 300 permises the bulk of the leakage has been localised in or just outside fifteen. To each of these premises he goes with the knowledge that a portion of the total leakage of 2,000 gallons an hour is almost certainly there, and that it must be found, which is a very different thing from visiting three or four hundred houses, is not one of which he has any particular reason to expect to find leakage. Even when he enters a house with previous knowledge that there is leakage, its discovery may be difficult. It is often hidden, sometimes underground, and may only be brought to light by excavation. In these cases, without some such system of localization, the leakage might go on for years or for ever."

The location of leaks in a distribution system can also be performed in a similar way, but for the purpose of this, it is necessary to arrange sluice valves in such a manner that different portion of the distribution pipes can be cut off as required. One meter is used for a block population of 2,000 to 3,000 persons, and the area is varied according to the density of population irrespective of the length of the main. The best time for inspection is about midnight, when the flow in a particular portion of the pipes is practically equal to the leakage in that portion, as the quantity actually drawn for use is negligible.

To carry out the test an inspector, after fixing a diagram paper on the rotating drum and closing the valve C (Fig. 131) on the main and opening the bye pass valves A & B, proceeds to the end of the district with a watch keeping the same time as in the meter clock. As before, it will be seen that the pencil on the drum traces a straight line on the diagram shewing the total amount of leakage in the block per hour. The inspector after reaching

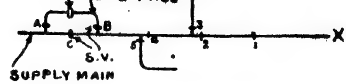


Fig. 131—Bye-Pass.

the end closes the valves in the order indicated by the numerals and notes down the time of each closing. After all the valves have been closed down, the inspector starts re-opening the valves in the reverse order until all are opened. The drop in the quantity of water passing in different hours as indicated in the diagram indicates the amount of leakage in the portion closed down. Thus, when the valve 1 is closed, the drop in the diagram, if any, will indicate leakage between 1 and X, and the drop after closing of valve 2 will indicate leakage between 2 and X, and so on. The leakage between 1 and 2 will be the difference between the drops in the first and second period.

If a meter is fixed in every block, and the supply is given through meter only, and a 24 hour diagram is taken every day, then the following information will be available from it.

(*i*) The quantity delivered during 24 hours and the rate of delivery during different hours of the day.

(*ii*) The lowest horizontal line in the diagram during the night hours indicates the quantity wasted through leakage of main and taps.

Deacon's waste water meter has given gratifying results in various towns in the Great Britain, and the percentage of waste reduced in many cases amounted to 80. By the use of this meter only leaks due to defects in the supply, distribution or service mains and their fittings can be detected, but it has no control over the waste due to individual carelessness or ignorance, such as opening of stop cocks for longer period than they are required, running to waste water collected in cisterns or vats, or lavish sprinkling of courtyards during day. In some towns, it has been estimated that the losses due to this item alone amount to 15 to 25 per cent of the daily consumption. Undoubtedly, the most rational way of preventing such waste of water is by fixing a meter on every service pipe and giving the supply through it only, and making each consumer pay for what he uses. This is the most equitable method of realising the cost of extra water supplied. If water is not supplied through meters, the careful users have to pay for their own supplies and a considerable

portion of the water wasted by his neighbour. If meters are used, the former pays only for the water he uses, and the latter pays for what he uses and wastes, just as it should be.

The general introduction of meters, both for the prevention of intentional or careless wastes in the consumers' premises or for the leaky and defective supply or distribution pipes and fittings, is extremely desirable, especially in this country, where many municipalities hardly have sufficient income to meet the annual running cost. By this means, the cost of operation of the plant will be considerably reduced, and a better pressure will be maintained in the distribution pipes, and the water will be supplied at low price and consequently, the water rate will also be low. It is sometimes contended that the universal use of meter is much too expensive both in initial and maintenance costs. But this seems to be a fallacy. For, if water is sold by meter measurement right from the beginning, the people will be more careful and the careful habits will soon become the second nature, which will prevent waste to such an extent that the savings in the cost of operation will in a few years' time repay the cost of meters.

In this connection, it will not be out of place to mention that in England water supplied for domestic purposes is invariably supplied at a flat rate, irrespective of the quantity of water consumed, based on the annual or rateable value of the holding. In Germany, the compulsory use of meters was adopted in many towns, but the consumption is not restricted below a desired quantity.

There is some difference of opinion as to who should provide for the supply and fixing of meters, the Municipality or the consumers? In many places, it has been contended, that meters are fixed on the requisition, and for the protection of the interest, of the municipality; as they serve to increase or maintain their revenue, they should pay for them. Besides, the careful consumers consider it an unnecessary imposition that they should pay for the meter, which is put in only to prevent the wasteful habits of their neighbours. We think, there is some sense in these

arguments, and instead of the meters being purchased by the consumers, it will be more satisfactory for the municipalities to buy them in quantities for the consumers. This will enable them to put in the most durable and efficient type of meters available and at a much less cost than that at which the consumers could supply them. The municipality by this arrangement will have to carry out repairs to parts for one make of meter, and consequently the cost of maintenance will be less, and meters will be repaired more readily and more efficiently. The municipality, which allows the consumer to purchase and maintain his own meter, also allows him *ip so facto* generally to take much of his water for nothing. This is what we gather from our experience in Bengal. Experience in America is not different. They have found the system to be unprofitable in the extreme, and a change to the method of ownership of meter by the suppliers declared absolutely necessary.

Summary of Methods of Waste Prevention—The method of preventing waste may be summarised as follows :—

1. The distribution system is to be divided into blocks supplying 2,000 to 3,000 population.
2. Sluice valves are to be arranged in such a way that all branches and parts of mains can be easily controlled.
3. Each block is to be supplied through a waste water meter.
4. Night inspection for detecting and locating leaks is to be done periodically.
5. Service connections and all works in connection with streets mains should be done by only authorised plumbers.
6. All fittings should be to an approved specification and stamped before use. Samples of all fittings should be kept in office for inspection.
7. A house-to-house inspection during day to examine the condition of fittings and to locate leaks is to be done.

8. Prompt rewasherings of taps. Inspectors should re-washer all the taps on their rounds when they find them leaking. This should be made free of charge.
9. Metering every service connection and charging the extra quantity supplied.
10. The initial cost of meters should be paid by the municipality, and for subsequent maintenance and repairs, a quarterly fee may be charged. Only one or two types of meters, which have been found to be satisfactory, should be used in one work.

Meters

The apparatus for measuring water under pressure is usually termed a meter. Meters are used both in connection with the distribution and service pipes, and are essential for the regulating and controlling of the supply and preventing waste.

Perhaps, no instrument has received greater attention than this instrument for measuring water. Every waterworks engineer is anxious to obtain the most perfect and reliable meter suitable for all purposes and capable of working under all conditions. With all ingenuity displayed and money spent, no instrument has been invented which satisfies all conditions of working.

Peculiarities of Meters—An ideal meter should satisfy the following requirements:—

(i) It should accurately register all water flowing through it from a dribble to the heaviest rush.

(ii) It must work under all conditions of pressure without losing efficiency.

(iii) It should absorb minimum amount of head in working.

(iv) Its cost of maintenance should be as low as possible, and it should be easily repairable without disconnecting the body.

(v) The parts should not be acted upon by the ordinary chemical and physical impurity in water, or those

used for its purification. The meter should not be liable to clogging.

(vi) It should prevent back-flow passing through it, and should not register when no water is passing through it.

Classes of Meters—There are meters of large number of makes in the market, and it is often very difficult to make a choice between them. Besides, meters can be had of different types, such as the *positive, semi-positive* and *inferential.* This makes the matter of selection of the best meter more complex.

In a Positive Meter, the quantity of water actually passing is measured by filling and emptying the chamber of a known capacity, which has been checked by actual weighing and measuring of the water. In the case of the Kennedy Positive Meter, which is the best known and most accurate of this type,

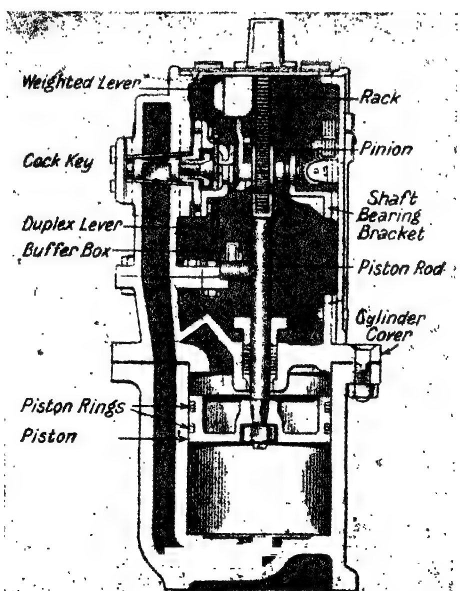


Fig. 132—Positive Meter.

(Fig. 132) it is the distance travelled by the piston rod which measures the quantity of water—not the number of strokes.

For this reason, the Kennedy Meter gives the exact quantity of water which has passed,—no matter at what position the cylinder of the meter stops. The delivery capacity of this meter is larger than that of any other, and it is of a very robust design. It has the advantage too of being correct at all speeds and pressures, and it does not matter how slowly the water may be passing in the pipe it is registered. For accuracy of measurement, there can be no doubt as to the superiority of this type of meter over the rest. But the cost of this type of meter is often 4 or 5 times the cost of an inferential meter, and about 3 to 4 times the cost of disc meter.

Inferential meters are operated on the well-known turbine principle. It consists essentially of a drum to which is attached a series of vanes or turbines revolving on a horizontal axis when rotated by the flow of water through the drum. At each revolution of the drum, a certain quantity of water is discharged, which is recorded by an indexing gear of wheel and pinion, and shewn on the dial on the top. This instrument in reality measures the velocity of water passing and not its volume. This type of meter is gauged and calibrated with fineness and works fairly well, within certain limits of pressure and flow, and with a very small percentage of error.

Inferential meters are much cheaper than positive meters, less bulky and work with a little loss of head. They are, however, less accurate and are of little use in measuring fine and intermittent flow. They should not be used, where there is liable to be air in the pipes. Another drawback of this type of meter is that if any thing causes them to stop working, the water will continue to flow through the meter though the quantity will not be registered.

Disc meters work practically on the same principle as inferential meters, although the design is somewhat different. Fig. 133 shows Trident disc-meter. The disc is flat and water balanced, eliminating friction. The gears of this type of meter are immersed in an oil-enclosed gear train which, it is reported, practically eliminates wear, corrosion, clogging

or deposits on gear trains. The disc type or semi-positive, as they are frequently called, gives generally better results, and are commonly used in preference to the ordinary inferential meters by some engineers. Apart from durability, permanence of registration and economy in the first cost and maintenance, these meters possess a peculiar advantage over other inferential meters. Mr. Hill found that a noticeable

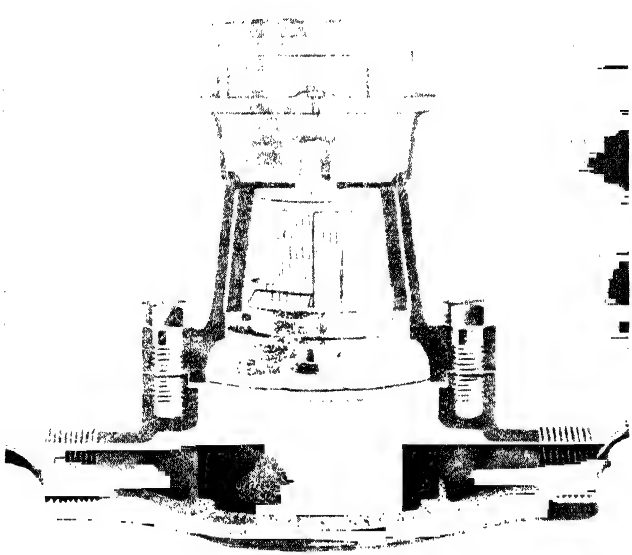


Fig. 133—Trident Disc Meter.

vibration of disc and attached spindle in the meters occurred with rate of flow due to small leaks in the service pipes and fittings, which were sufficient to cause registration. The accuracy of this meter depends upon the closeness of fit of the disc, but at the same time it must have sufficient play to prevent jamming.

Now, it may be asked whether the difference in accuracy and the cost of maintenance warrant the use of positive meters in preference to others. After careful investigations, many engineers have come to the conclusion that a fairly large per-

centage of error can be allowed before the loss becomes equal to the extra cost of positive meters.

The following types of positive and inferential meters are generally used in waterworks :—

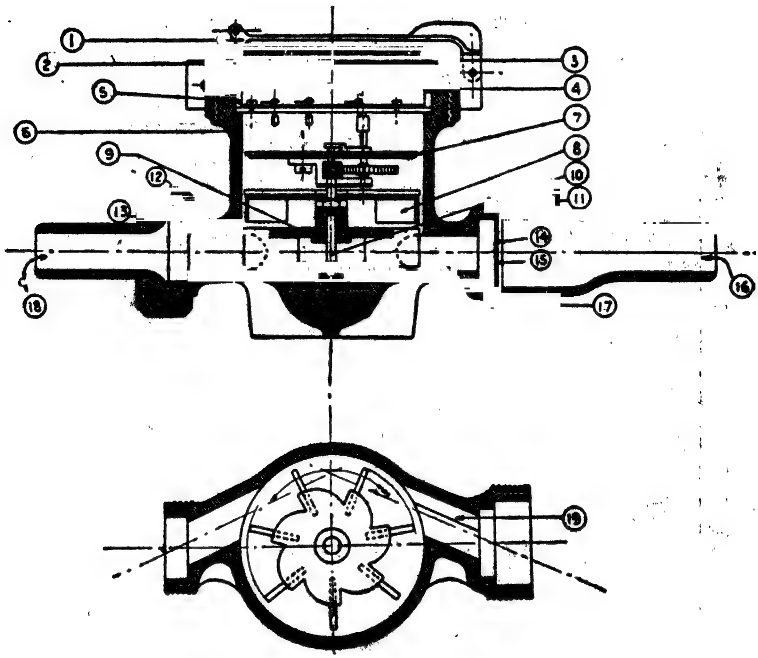


Fig. 134—Rotary Meter.

1. Index door.
2. Glass disc.
3. Cover.
4. Rubber joint.
5. Distance piece.
6. Body.
7. Index.
8. Adjusting blade.
9. Driving vane.
10. Bearing spindle.
11. Nut for inlet tail piece.
12. Nut for outlet tail piece.
13. Leather washer for outlet tail piece.
14. Copper strainer.
15. Spring ring.
16. Inlet tail piece.
17. Leather washer for inlet tail piece.
18. Outlet tail piece.
19. Patent dividing partition.

Positive Meters—

(1) Reciprocating piston, one cylinder type (Fig. 132).
(2) Reciprocating piston, two cylinder type.
(3) Rotary piston type.

Inferential Meters—

(i) Rotary fan (Fig. 134).
(ii) Rotary turbine (Fig. 135).
(iii) Ventury Meter (Fig. 138).
(iv) Waste water meter (Fig. 139).
(v) Pitometer (Fig. 140).
(vi) Disc Meter (Fig. 133).

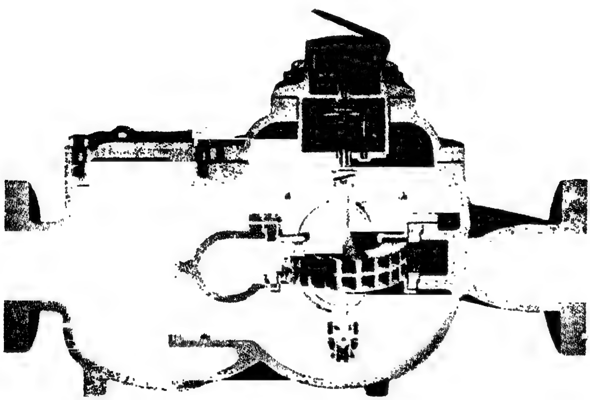


Fig. 135—Turbine Meter.

Of these, rotary, turbine and disc meters are generally used in service connections.

Venturi meters are used in trunk or district mains, either to measure the flow or to detect waste. Positive meters are used in residential institutions, restaurants, public houses etc., where the supply is large and a more accurate measurement is required. Positive meters have also been used in service connections in many places, especially where water is expensive.

The size of meter to be adopted in a particular case should be determined on the basis of the amount of water used and the rate of flow rather than upon the size of service pipe in which it is to be fixed. Only such type of meter should be used, as can work without much loss of head under the minimum pressure available at the place.

The limit of error of registration usually specified is 2 per cent fast and 3 per cent slow, i.e., the meter must not register more than 2 per cent in favour of the supplier, nor more than 3 per cent in favour of the consumer. Long experience and careful investigation shows undoubtedly that all meters become slower as they get worn and out of repairs, as such condition increases the *slip*. Mr. L. R. Hawson in a paper read before the American Waterworks Association gave statistics which clearly shows that the average percentage of accuracy of 20 per cent of meters in many waterworks varies from 22 to 40, and the other 80 per cent registered from 97 to 100 per cent accuracy. Generally speaking, meter used for measuring filtered waters will register more accurately than those for ground waters.

In Johanesberg, 24,000 disc meters were used in service connections and maintained in efficient condition by the waterworks department. The regulations in force, as reported by the town engineer Mr. A. S. Burt Andrews, were briefly as follows:—

All meters are the property of the council. Meters when desired are fixed in a suitable and safe place provided by the owner. Consumers are responsible for the safe keeping and condition of the meter. Consumers are not allowed to interfere with any meter, or cause any other person to do so. Meters are maintained and repaired by the council, only the cost of damage caused by interference or accident is charged. A meter is taken as correct, if the error is not more than 2½% either way. The meter is tested by the council in case of dispute, and a charge is only made when the meter registration is found correct. When a meter is out of order, the charge is based on

previous consumption. If the charge for water is not paid within a reasonable time, the supply is cut off and a fresh fee is charged for reconnection. A monthly rent of meter is charged for a temporary supply and the meter is given to the party after depositing its value, on condition that it will be returned in good condition, or the party will pay for the cost of repairs, if damaged in any way during the period of the loan.

Meters should be tested both for accuracy and durability before being placed in service, and where large number of meters are actually in use, suitable testing apparatus should be provided for the purpose. More elaborate meter tests are carried out on the continent of Europe than in England. There the test is carried out as nearly as possible under the same working condition, the same length and size of service pipe as generally used and under ordinary pressures. The apparatus generally consists of large tanks fixed on weighing machines, so that the water passing can be measured in volume and weight at the same time. The tank is accurately calibrated and fitted with a gauge glass to show the contents of the tank at different heights. The meter is attached to such tanks placed at different levels. The meter is tested first for accuracy of registration and measurement, before it is subjected to any work. Then, it is put on an endurance or durability test, during which the meter is made to work for a term in which it will pass about as much water as in ordinary circumstances would flow through it in a year. Then, it is subjected to a test of intermittent flow. Finally, after these tests, it is tried for accuracy again, and in the end the parts are taken to pieces and the condition of the wearing surfaces are carefully examined.

In this country, the test adopted are not so elaborate, only the test of registration and accuracy are made by connecting the meter to a supply main, and arranging it to deliver into a calibrated tank fitted with a gauge to show capacity at different height. The delivery in particular period as indicated in the meter is compared with that delivered into the tank. Figs. 136 and 137 show the stationery and portable meter testing apparatus.

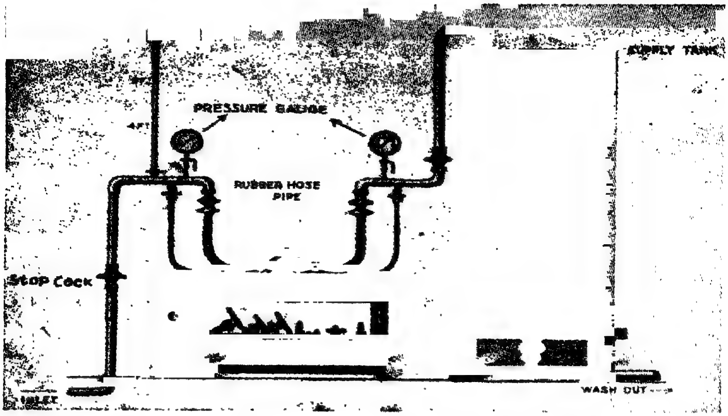


Fig. 136—Stationery Meter Testing Set.

Fig. 137—Portable Trident Meter Testing Set.

Venturi Meter (Fig. 138)—This is a very valuable instrument for measuring flow of water in large pipes and with a nominal loss of head. It cannot, however, measure small flows, the range of registration ordinarily being 14 to 1 and with a special recorder, it can be increased to 20 to 1. The accuracy

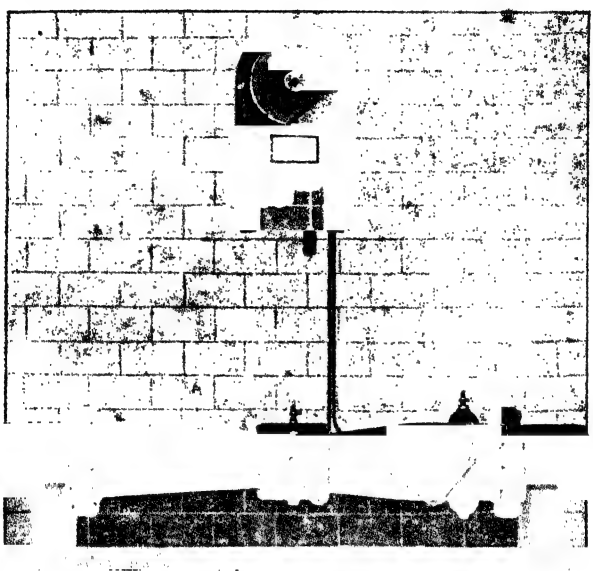


Fig. 138—Venturi Meter.

of registration of this meter is remarkably high being on an average of 95·5% of the flow. This is generally used on pipes from 6 inches up to 10 ft. in diameter and for any pressure, any velocity, any volume and any kind of fluid. It is not injuriously affected by water hammer nor by the most violent fluctuation of pressure or velocity.

This meter is named after the Italian philosopher, Venturi, who discovered that when water flows through a converging cone and a narrow throat and then expands into a diverging cone, there is a decided decrease of water pressure inside the conduit from the beginning of the converging cone to the

venturi (throat) at which point it is minimum, and there is a decided increase in pressure from the end of the throat to the end of the diverging cone. Subsequently, the American hydraulic engineer, Hershel, has clearly demonstrated that the difference in pressure at the beginning of the mouthpiece and at the venturi increases with the velocity of flow and has established by numerous experiments the relations between the two, so that by knowing the difference of pressure at these two points the discharge can be determined.

The tube forms a part of the pipe line on which it is fixed, and only differs from it that it presents for a short distance a truncated converging cone coupled by throat piece to a similar expanding cone. The relation of area of the throat to that of the main in which the tube is inserted varies according to the maximum and minimum registration required, the area of the throat being increased when the ratio of registration is high, and decreased when the ratio is low. The length of the converging and diverging cones are generally made 17·09 and 69·80 times the length of the venturi tube, but this ratio is also varied by several markers.

The recorder, broadly speaking, consists of two parts, first of a mercurial U tube which being connected with the up-stream and throat brings in the elements of venturi head, and secondly, clockwork and the gear controlled thereby supplies the element of time. The connection between the pressure and time is established by means of floats resting on mercury in the U tube. The recorder is generally fitted with a diagram arrangement, showing the rate of flow as well as with a count which gives the total quantity passed during different hours. It can also be fitted to show the pressure.

Deacon's Waste Water Meter—This is shewn in Fig. 139. The instrument essentially consists of a gunmetal tapered tube within which a gun metal disc is suspended by means of a wire passing through a packed gland and connecting a small carriage for pencil moving vertically between guides. A cord secured to the upper part of the pencil carriage passes over the pulley

and supports the counter balance weight at its other end. The drum is revolved horizontally by an eight day clock. When there is no flow through the pipe, the disc is raised by the counter balanced weight to the top of the truncated cone with

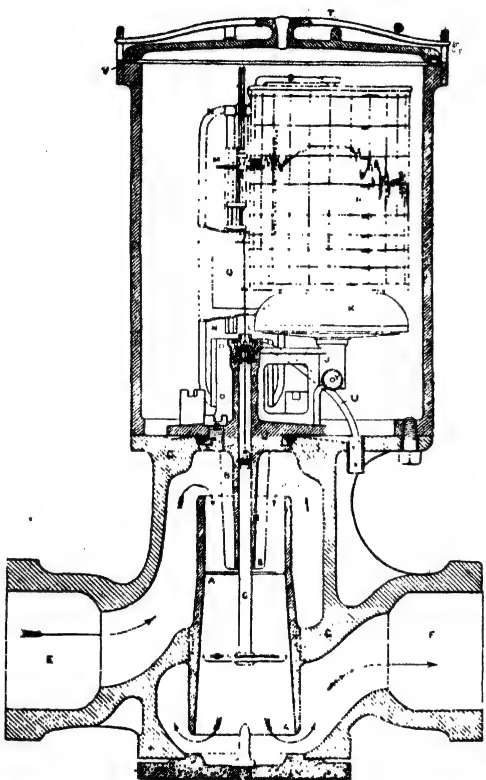


Fig. 139—Deacon's Waste Water Meter.

which it fits exactly and the pencil points to the zero line of the diagram. As soon as a flow occurs, the disc is moved down into the cone in proportion to its volume, and the vertical position of the pencil is altered corresponding to the movement of the disc, and a line is traced on the drum to represent the amount of flow at different hours of the day. It may be placed directly on the main or a byepass controlled valve.

The use of this meter in detecting waste has been explained at the beginning of this chapter.

Pitometer—A portable pitometer is shewn in Fig. 140; it is in reality a current meter used for measuring the flow of water in pipes and open channels. It is designed on the principle of *pitot tube,* i.e., if a bent tube is inserted in a stream of water having a velocity with open end of the bent squarely facing the direction of flow, then theoretically the water will rise to a height in the tube equivalent to the square of the velocity of the current divided by 2g.

The pitometer consists of two parts, the *Rod meter* and *Manometer.*

The Rod consists of a brass sheath containing ¼ inch tubes, each of which terminates in bent orifice of phospher bronze; the orifices are turned out in line parallel to the flat side of the sheath, so that one is directly opposite to the direction of flow and the other parallel to it. The manometer is an arrangement for measuring the velocity head indicated in the pitot tubes. This is effected by means of a glass U-tube connected by rubber tubing to the brass tubes at the top of the rod meter. The U-tube is provided with a scale divided to foot and tenths. The lower half of this tube is filled with coloured fluid not soluble in water, and having a specific gravity greater than that of water. The upper half of the tube and the connecting tubes are filled with water. When the orifices are inserted into a current of water, the velocity causes the coloured indicating liqhid to rise in one leg and fall in the former, and the vertical distance between the surfaces of the indicating liquid represents the velocity head from which the flow is measured. But, owing to the fact that the distribution of velocities in the cross section of a pipe with flowing water is affected by upstream disturbances and by the roughness of the interior surface of the pipe in places where precision is required, a complete traverse should be made. The ten point method of making traverse consists in observing the velocities of ten concentric circles dividing the cross section of the pipe into 10 equal areas. The velocities are taken on the same diameter, and they are

added up and divided by 10. The theoretical mean of velocity is 0·30 too high, and may be neglected for all practical purposes.

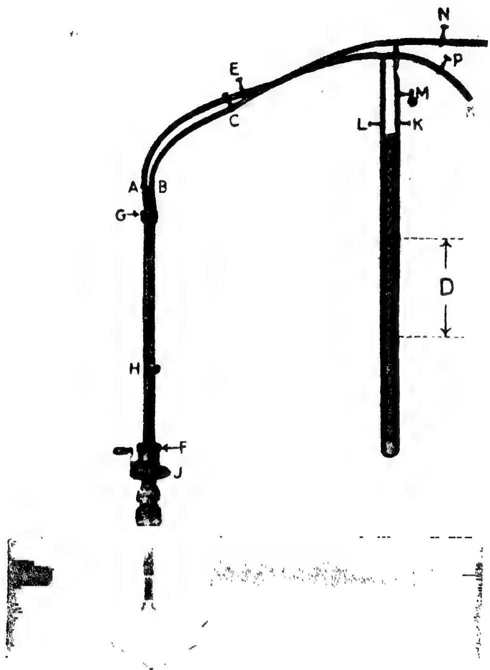


Fig. 140—Pitometer.

The makers usually issue instruction books for the use of such meter, giving table of co-efficients of the instruments with various velocities. They are indispensable in using such meters. This instrument is a ready means of gauging the flow in a pipe without going to the complexities of fixing a positive or inferential meter. All that is necessary in this case is that a hole (1 inch diameter) is to be drilled and tapped at any point in a main, a plug cock is fixed and the instrument is inserted through the cock, the quantity of water passing can be at once determined, if a traverse of the pipe has already been taken. It is obvious that by inserting the instrument at successive points of pipe line, the difference in flow can be determined

which may be the consumption between those points or leakage ; if the flow is measured between 1 to 4 A.M. when the majority of the consumers have retired, and actually the quantity being drawn for domestic use is infinitesimal, it will represent approximately the leakage in the mains and service pipes.

CHAPTER XIV.

MANAGEMENT AND MAINTENANCE

Financial —In the preceding pages an endeavour has been made to explain the general principles of water supply engineering in special reference to conditions prevailing in this part of the world. There remains for consideration the subject of financing the initial cost of the scheme, and also the cost of operating it. When a public water supply is owned and operated by a private company, as is the case in many cities in the western countries, it is conducted on ordinary business principles, and is only undertaken when the investor's interest is fully guaranteed. But, in the case of municipal ownership, besides the cost of operating the plant, the local body is faced with an additional problem of meeting the cost of construction. The fundamental principle followed in this connection in other countries is that these works must be self-supporting, and should not be called upon to support other branches of municipal services, or to produce profits to be used in reduction of general taxation, or for such purposes in which the present generation is not directly benefitted.

No town can pay cash for such works from its treasury. As all works of this nature is undertaken not only for the benefit of the present population in the town but also to a considerable extent for the future generation, it is but fair and just that the latter should pay as much for the service rendered as does the present generation. This object is attained by raising a loan for the initial capital cost of the works, such loans extending over periods during which the plant can be usefully utilized without additional cost. The period has been limited to 30 years in Bengal by a committee appointed by the Sanitary Board.

The borrowing capacity of the municipalities in Bengal is very limited. The average incidence of income is slightly less than Rs. 4/- per head per years, of which only Re. 0·66 is available for works of this nature. The initial cost of a scheme

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MANAGEMENT AND MAINTENANCE

Financial —In the preceding pages an endeavour has been made to explain the general principles of water supply engineering in special reference to conditions prevailing in this past of the world. There remains for consideration the subject of financing the initial cost of the scheme, and also the cost of operating it. When a public water supply is owned and operated by a private company, as is the case in many cities in the western countries, it is conducted on ordinary business principles, and is only undertaken when the investor's interest is fully guaranteed. But, in the case of municipal ownership, besides the cost of operating the plant, the local body is faced with an additional problem of meeting the cost of construction. The fundamental principle followed in this connection in other countries is that these works must be self-supporting, and should not be called upon to support other branches of municipal services, or to produce profits to be used in reduction of general taxation, or for such purposes in which the present generation is not directly benefitted.

No town can pay cash for such works from its treasury. As all works of this nature is undertaken not only for the benefit of the present population in the town but also to a considerable extent for the future generation, it is but fair and just that the latter should pay as much for the service rendered as does the present generation. This object is attained by raising a loan for the initial capital cost of the works, such loans extending over periods during which the plant can be usefully utilized without additional cost. The period has been limited to 30 years in Bengal by a committee appointed by the Sanitary Board.

The borrowing capacity of the municipalities in Bengal is very limited. The average incidence of income is slightly less than Rs. 4/- per head per years, of which only Re. 0·66 is available for works of this nature. The initial cost of a scheme

to give only a minimum supply of 5 gallons per head of population will be not less than Rs. 17/-, and the annual running charges will be about Rs. 0·61 per capita. If the whole of the initial cost is met from a loan at 5% interest, then the annuity will be Rs. 0·26 per head per year. Then the total annual charges will be Rs. 0·61 + 0·26 = 0·87 per head, which is beyond the financial means of local body generally. The Government of India, however, recommended that the local governments should contribute at least a third of the cost of such schemes and grant as much loan as the local body could repay. Even with the third of the cost from Government, the municipalities cannot meet the balance required for construction ; consequently, they usually fall back upon private subscriptions for the remaining cost of the scheme.

The loan in this country is repaid in equal half yearly or annual instalments, which corresponds to a similar deposit in a sinking fund. A sinking fund technically means a fund built up during a period of time to make provision for a sum at the end of that period by depositing equal sums of money at equal intervals in a place, where these deposits can derive compound interest. The amount of deposit depends upon the rate of interest, and the interval at whirh it is paid and also the period of investment. The amount accumulated in the sinking fund, therefore, is the sum of the amounts of deposits at the end of the life of the fund together with the compound interest accumulated.

In schemes owned and operated by a private corporation, a depreciation fund is necessary to furnish capital for renewal or replacement of the whole of the plant or portion thereof, otherwise the business will have to be wound up when the plant comes to the end of its life. But, in case of water works owned by local body, there is hardly any justification of such a fund. It is a marked injustice to the present population to compel them to accumulate a fund for the construction of a new plant of which a future generation will enjoy the benefits. It is still more undesirable to require the present generation of Bengal to contribute towards the benefits of others, when they

cannot themselves carry out waterworks without begging and borrowing. It is a good policy if the original debt including that portion covering the permanent parts of the works is fully paid off, then this will turn over to the future generation a property free from debt, but a portion of which in a more or less deteriorated and inefficient condition ; but that generation can use its taxing power or other financial resources of the town to renew, extend or enlarge the plant as may be required. There is also hardly any ground to believe that the future government of the country will deprive the new generation of the privilege of government contribution for such works, or that the new generation will be less patriotic and charitable in raising funds for works for their own benefit. For similar reason, the expenditure on military and naval works in the advanced countries are generally met from national loan, as the future generation are equally benefitted by them, so they are made to bear an equitable share of the burden.

There remains another matter to be dealt with in this connection, namely, provision for renewal of parts of the plant such as tube well, or worn out parts of the plant, the life of which is comparatively short and the breakdown of which will stop the supply to the town altogether, and thereby affect the financial resources of the local body. If the waterworks is properly managed and maintained in a reasonably good condition, there should not be any difficulty in meeting the extraordinary cost of this nature. In fact, in one or two municipalities in Bengal, it has been possible to borrow money from water works fund for other purposes. Even, if this is not possible in all cases, the water rates under the municipal act are liable to revision every three or five years, and each revision brings increased revenues ; it should, therefore, be quite within the financial resources of the municipality not only to undertake such emergent works but to make reasonable extension necessary to meet the requirements for subsequent development. In England, such cost of renewals and repairs is however included in the annual working cost of the plant. The rates must be so fixed that the receipts will be sufficient

to meet the maintenance expenses of the work, and the annual instalment of the loan raised for the construction of the works.

Useful Life of Different Parts—The life of a waterworks as a whole has been assumed in Bengal to be thirty years, and on this basis loans granted to local bodies are made repayable in 30 years. But the actual life of a plant or any portion thereof can seldom be estimated with any exactness. A plant may outlive its period of utility, and at the same time may become altogether of no service to the installation. Besides, the life depends considerably on the way, in which a particular plant is erected and the care taken to maintain it. The following useful life is given by Leonard Metcalf in the Transactions of the American Society of Civil Engineers, 1909, and generally accepted by engineers in other countries.

TABLE 57.

	Years.		Years.
Impounding Reservoir	50 to 100	Hydrants	40 to 50
Elevated Reservoir ..	26 ,, 40	Sluice Valves	40 ,, 50
Masonry buildings ..	40 ,, 50	Pumping & auxiliary machinery ..	20 ,, 30
Cast iron pipes of large dia.	50 ,, 75	Steam Engines ..	15 ,, 25
Cast iron pipes of small dia.	20 ,, 40	Boilers	12 ,, 16
Steel pipe	25 ,, 50	Electrical Machinery ..	20 ,, 30
Service pipes	15 ,, 20	Filter beds	30 ,, 50
Meters	20 ,, 30	Mechanical filters ..	20 ,, 40

Working Cost—The annual working cost of a waterworks is usually composed of the following items:—

1. Labour.
2. Materials and supplies.
3. Rents, taxes, insurances etc.
4. Instalment for repayment of loan.
5. Contingencies.

LABOUR—This item includes expenditure on labour of all kinds whether mental or physical. It includes salaries paid to (i) superintendents, drivers, firemen, oilers and other pumping

station establishment; (ii) the staff necessary for the upkeep of filer-beds and distribution system; (iii) store-keeper; (iv) office expenses, etc., and other expenses incidental thereto.

MATERIALS AND SUPPLIES—This includes all materials necessary for (i) operating the plant such as oil, fuel, chemicals etc., and (ii) maintaining the system in good condition, such as spare parts, lead, packing etc.

RENTS, TAXES, INSURANCES, AND INSTALMENT FOR REPAYMENT OF LOAN—These include all charges payable to Government, other local body or the landlord, if any.

In brief, all expenditure incurred in paying the wages of the staff employed for running the waterworks, pumping and purification, and in maintaining the system in a good and efficient condition is included in the annual maintenance charges of the scheme.

Water Rate—The money, required for maintaining the works in good condition and for operating it, is obtained from the levy of a water rate on the consumers within the municipal limits, and also from sale of water to other bodies. The water rate is really the value of the water actually or presumably supplied to a consumer in his premises for domestic purposes or for public purposes, such as drain-flushing, street-sprinkling etc. for the general well-being of the community. This is not really taxation, but value realised for a commodity sold, and is often cheaper than water that can be obtained by other means. The water rate in this province is limited to 7½% of the annual valuation of the holding, and is seldom below 3%. The water rates are generally divided into two classes, according as the water is supplied by street standposts or house-connections. The former should be based upon the water presumably used, and the latter on the quantity actually used, as the latter gives the opportunity of practically unlimited use of water and of wasting it. If the actual quantity in the latter instance is not paid for by the consumer enjoying the luxury of wasting the water, then the poorer population drawing a limited supply from standposts will in one sense be taxed iniquitously to pay for the

luxury of his rich and influential neighbour. But, unfortunately, such is the case in almost every town in Bengal, as the excess water is seldom charged for by the local bodies. The chief objections of local bodies to not measuring the extra water consumed and charging them are:—

(1) That standpost being fixed 300 to 400 feet apart, the neighbours of the persons having house-connections usually draw all their supplies from the house-taps instead of from standposts. This is especially the case, when the water for the household is generally drawn by the maid-servants or female members of the house. They feel shy in going to the crowded standposts for water.

(2) The meters do not work satisfactorily and often get out of order.

There is some sense in both of these objections; a Hindu cannot on any account refuse the use of his tap to his neighbour. It has also been found by experiment that about 50 per cent. of supply to dwelling houses is drawn at an average rate of not more than 8 or 10 gallons an hour and a considerable portion of this is taken merely at a dribble. In such conditions, inferential meters, which are generally used in service connections, do not work satisfactorily. This is especially the case where the working head is very small. We think, however, this difficulty can be removed by increasing the residual head at the end of the distribution to at least 20 ft. above ground, syphoning the water through the meter, by placing it at a level lower than the communication pipe, so that a meter always remains full of water and also by making some air-escape arrangement before the meter.

This air difficulty can also be considerably avoided by the use of disc meter in service connections.

Another objection has been put forward in this connection, that it is very difficult to maintain the meters in working condition. This is entirely due to the fact that the work of repairs being left in the hands of some untrained labour, and the difficulty is enormously increased by the municipality

allowing the consumers to use all types of meters available in the market. If the municipality allow the consumers to use only one type of meter, then it will have to stock spare parts of one kind of meter, and the task of repair will be comparatively easy. Meters repaired in the present way cannot but get out of order. Further, if the responsibility of meter repairs rests with the municipality, the difficulty will be further reduced.

In other countries in several places meters are supplied by the undertaking firm or the municipality, as the case may be, and all meters are maintained and repaired by them, for which a quarterly fee is charged. This is a sound scheme, and should be followed by the municipalities in this country. They can easily include the first cost of supplying and fixing meter in the fee usually charged for allowing house-connections. By this arrangement the municipality will be free to use one and the most suitable type of meter and stock a comparatively small number of spare parts, and their meter repairing staff will pick up the work of repairs very quickly and do it more efficiently.

Meter Rate—Meter rates vary with the quantity of water supplied. There is always a free allowance made to the consumer in respect of the amount of water-rate paid by him, and the water in excess of the free allowance is charged according to a graduated scale of rates. The scale should be framed in such a manner that it offers no inducement for the consumers to waste water and to realise the benefit of a lower rate.

The following are the model rules followed in Bengal in this connection :—

(1) Every owner or occupier of any holding, in respect of which a connection has been made, under these rules, shall be entitled to a supply of—

(a) a fixed number of gallons of water per quarter for each rupee, and

(b) a further proportionate number of gallons per quarter for every additional fraction of a rupee,

paid by him quarterly as a water-rate in respect of such holding.

(2) For all water in excess of the amount allowed under sub-rule (1), such owner or occupier shall be charged quarterly as follows:—

	Annas per 1,000 gallons.
(*a*) For any excess quantity of water not exceeding one-half of the amount of the allowance prescribed in sub-rule (1)	...
(*b*) For any excess quantity of water exceeding one-half of the amount of the allowance prescribed in sub-rule (1), but not exceeding that amount	...
(*c*) For any excess quantity of water exceeding the amount of the allowance prescribed in sub-rule (1), but not exceeding twice that amount ...	...
(*d*) For any excess quantity of water exceeding twice the amount of the allowance prescribed in sub-rule (1)	...

The meter rates are applicable to water used for "domestic purposes". The term "domestic purposes" is very indefinite and vague both in English and Indian law, and has been the subject of severe criticism on the part of the judges in England. Section 12 of the English Act 1863 defines *domestic purposes* as follows:—

"A supply for domestic purposes shall not include a supply of water for cattle or horses or washing carriages, where such horses or carriages are kept for sale or hire or by a common carrier or a supply for any trade, manufacture or business or for watering gardens, or for fountains, or for any ornamental purposes".

Rate for Water Closet Supply—The above section does not exclude baths or water closets from domestic purposes. The clause in the Model Water Bill, however, provides for the payment of charge not exceeding five shillings per annum in respect of every water closet beyond the first on the premises. These charges varied very widely in different undertakings. The following summary of practice is taken from Parsons' "Waterworks Administration":—

(*a*) First free— each additional at rising rates.
(*b*) First and second free—each additional at fixed rates.
(*c*) First free, second at fixed rate, additional ones not charged.
(*d*) First at rising rates—each additional at fixed rates.
(*e*) First at rising rate, each additional at half rates.
(*f*) Up to £12 first free ; each additional at fixed rate : above £12, first and second free ; each additional at fixed rate.
(*g*) Up to £6 first free : above £6, first rising rate ; each additional at fixed rate.

The scale of charges has all been authorised by Parliament.

The above information is given in some detail, as with the introduction of sewerage systems in some of the mofussil towns of Bengal, it may be found soon necessary to fix up rates for water closets which are really a luxury of the rich, as the poorer generation have not sufficient means to provide for them.

UPKEEP

The problems, which arise in connection with the upkeep of a waterworks, are so varied and complex that they can only be solved by a careful, intelligent, failthful and experienced engineer. Works of this nature should never be left in the hands of a mediocre. In many cases, this has been the cause of loss and trouble a short time after the appointment.

In the next few pages, it is intended to notice only some of the common attention, which a good superintendent is required to pay for such works.

Impounding Reservoirs —The watershed should be maintained free from all pollution, whether direct or indirect. With this object in view, it should be kept free from all cultivation ; and rank vegetation should be cleared, removed and burnt, once before and after the rains.

The gutter round the reservoir constructed for the purpose of arresting the silt and garbage carried by the surface flow should be always kept cleaned and maintained to proper level. No cattle should be allowed to enter the streams feeding the reservoir when possible ; when this is found impracticable, at least a length of one mile of the stream should be kept free from such pollution. In short, every attempt is to be made to keep the reservoir and its watershed free from all organic impurities at all times. This does not of course apply to fish which help the purification of other organic impurities. If there be growth of algae or other vegetable matter, care should be taken to prevent them being taken into the conduit.

The sides of the reservoir should be cleaned at the time of its lowest level, and all growths on it are to be removed outside and burnt. All leaks in the dams should be repaired immediately they are noticed, as they generally increase in size and make rupture imminent. The other works of the reservoir and its accessories, the conduit, the valve tower, the waste weir etc., should be carefully looked after to see that they are in a sound working condition.

A careful daily record of rainfall and run-off, and quantity of water delivered to the consumers and the water level in the tank should be maintained at the station to see if any additional storage is necessary, or if the demand can be reduced by some method. If possible, an accurate measurement of the evaporation and absorption of water from the reservoir should be made and recorded in the station. A regular analysis of water should be made to control the bacterial efficiency of the supply.

Wells and River Intake —Intakes generally require little attention beyond removing the stoppages at the strainer or

round the jetty. Fish, sticks, cloth or other floating matters are sometimes caught in the strainer and have to be removed. But the supply may be dangerously affected, when any human or animal corpse is caught in the jetty or strainer. Great precaution should be taken to see that this does not happen.

The level of the bed of the well or the river should be periodically taken to see if it is rising, and if the suction pipe is drawing sand from the bottom. Sand entering a pump cuts the plunger and the cylinder surface, and increases the slip rapidly. Every means should be adopted to exclude, and when it is possible to do so, to intercept, sand before it reaches the pump. For this reason, the bottom of the suction pipe is kept well above the bed and generally only 3 or 4 feet below the lowest water level in the river or the well, so that the stoppages of the rose below footvalve by floating matters is reduced to the minimum.

Suction pipes should be frequently examined to see if there are leaks in any point, as the air from leaks causes the pump to "pound" and thereby considerably shorten its life.

Pumping Plant And Fuel—Pumping plants are generally provided in duplicate, so that one remains as standby. The efficiency of a well-designed pumping plant depends considerably on the capacity and faithful working of the superintendent in charge of the station, in addition to the quality of fuel and other materials and tools provided. Three kinds of fuel are generally used in this country, namely, liquid fuel, crude oil, and coal. Crude oil is practically the monopoly of one or two companies and is more or less of uniform quality. These oils have generally a thermal capacity of 17,500 B.Th.U. per lb. and a specific gravity of about ·87. Kerosine has a heating value of 18,000 B.Th.U. per lb. and a specific gravity of about ·8.

Average Bengal coal contains about 9,000 B.Th.U. per lb. and 15% ash. The coal used in boilers should be low in ash, content and moisture, since an increase in them is apt to cause falling of both in capacity and economy of the boiler. The coal should be free from dust as much as possible, and the

average size of the pieces should be about the size of an egg and should not be less than a pea. In selecting coal, attempt should be made to obtain only that which is clean, bright, conchoidal fracture, and which is free from slate and earthy matter. Coal, when exposed to the rays of a tropical sun, undergoes a change which reduces its evaporative capacity.

Boilers —The following may be useful to those in charge of boilers :—

1. Safety valves should be tried every day to see if they are in working order and not overloaded, and should blow off at the same pressure every day.
2. The steam gauge should indicate zero when the pressure is off and the same pressure when the safety valve is blowing off.
3. Water level should be at its proper height before starting. This is to be tested, both by means of a gauge glass and gauge cock, as the former is not always reliable.
4. Cold water should never be fed into a boiler, nor pumped into a hot boiler, during first filling. This has caused serious leaks, and even explosions in many cases.
5. Frequently, unsuspected waste happens owing to air leaks being not carefully stopped.
6. The boiler should be emptied and cleaned every month and filled afresh. Blow off cocks and check valves should be examined each time the boiler is cleaned. The feed water should be blown off as frequently as the condition of water required.
7. Care should be taken to keep the boiler-house floor dry and clean, as dampness tends to corrode and weaken metal plates. Every thing about the house should be kept clean and in good condition. Negligence in this respect tends to waste and decay. The ashes from boilers should be slaked outside

8. Feed pumps should be of ample capacity and be maintained in perfect order. No make of pump can be expected to work continuously without regular and proper attention.

It is always desirable to have two arrangements for feeding boilers ; check valves and feed valves should be frequently examined and cleaned.

With regard to the management of engines, indicator diagrams are very useful. In fact, in the hands of an experienced engineer, the indicator is as the stethoscope of the physician. It indicates if the valves are properly set or if the valves are leaky ; in short, it reveals the secret workings of the inner system, detects minute derangements in parts obscurely situated and registers the actual power developed by the engine.

Indicator diagrams should be taken at least once every week, and careful records of them should be maintained in office. Any defects indicated in the diagram should be forthwith attended to.

There are obvious losses in practice, which render the actual amount of steam used per horse power in excess of that calculated from the indicator diagram. These losses are by condensation and leakage. With proper care and attention they are preventable to a considerable extent. Leakage is an insidious evil ; a superintendent can hardly render better service to his employer than by keeping the piston valves and the glands tight. It has been tested on many occasions that losses due to condensation and leakage are one half of the steam used.

Lubrication—Friction is another loss of power, and it has been proved conclusively that, so far as the bearings of machinery are concerned, the true and practical definition of friction is want of lubrication. Some breakdowns have been caused by careless and insufficient lubrication and consequent rapid deterioration, and sometimes even a complete and instantaneous destruction of the working parts of the machinery. For the proper lubrication of the running parts, the chief

qualities to be looked for a lubricant are its capacity to reduce friction to the lowest possible degree, uniformity of composition, capability of enduring high temperature without decomposition, and low temperature without becoming solid.

Some of the methods used by engineers to specify lubricating oils are by their specific gravity, viscosity, flash point, burning point, cold test, acidity and friction test.

Different kinds of lubricants are necessary for different parts of the plant according to the pressure, speed and heat they are subjected to. It is always best to consult the makers of the machinery about the lubricant and use only the oil recommended by them.

Slip—The *slip* of the pump should be tested every week ; it increases with age and lowers the delivery capacity of the pump. The slip in a pump is not difficult to remedy, and should never be allowed to be excessive. Any negligence in this respect lowers the efficiency of the plant, and consequently increases the cost of pumping.

Economy of Running Cost—The economy of running cost is the chief problem in connection with the management of pumping station. Appendix VII gives the running cost of some of the waterworks in Bengal under different heads of expenditure. From this, it will appear that hardly any economy is practicable excepting stores and fuel. The staff employed is practically fixed by the size of the plant and its working hours. The chemicals used in coagulation and filtration will be the same, if the quantity delivered remains unchanged. The cost of repairs should be nominal, unless the superintendent is careless, or the plant is old and worn out.

In some waterworks in the west, a bonus is offered to the superintendent for every million foot pounds by which the duty exceeds a certain amount. This is a good practice, as the extra duty can only be obtained by more careful and efficient working of the plant. In this connection, we would suggest in steam plant stations a free allowance of coal should also be given to the establishment living in the station for domestic use.

Small Size Diesel Engines and their Operations.

Two or Four-stroke Engines—Two-stroke engines are equally reliable as four-stroke engines when without crank chamber compression but with scavenging pumps. Piston scavenging pumps are more efficient and durable than those of the centrifugal type. The fuel and lubricating consumption of two-stroke engines is generally slightly higher, and their lives somewhat shorter. These disadvantages, however, are compensated by a simpler design doing away with inlet and exhaust valves and their timing gear, with valve grinding and adjustments of the tappets.

Testing of Diesel Engines Before Starting—Engines just erected, not run for a long time or found defective, should be carefully examined before starting.

LUBRICATING DEVICE—Operate the automatic lubricator, open the sight feed lubricators or turn the engine when equipped with force feed lubrication system and make sure that all parts subject to wear and tear are supplied with oil. Oil filters, pipes and borings must be frequently cleaned.

BEARINGS—Crank shafts of multi-cylinder engines or engines with outer bearings must be specially lined up. Connecting rods must be exactly in right angles with the crank shaft, otherwise the small end bearing will be forced against the inner sides of the piston or the crank shaft starts moving to and fro.

The big end bearings of four-stroke engines must have a radial clearance of 1/500 of crank pin diameter ; in two-stroke engines, this clearance may be somewhat larger, because there is no change of pressure in their connecting rods.

COMPRESSION—Turn the flywheel against the compression and see whether air is leaking through inlet or exhaust valves, from between cylinder and head or along the piston. Leaky valves must be ground, and defective cylinder head packings renewed. If the compression is leaking along the piston, the latter is to be taken out and examined. Clean the piston rings

and grooves, distribute the ring locks all round, polish the cylinder liner and examine the lubricator and oil pipe. Piston rings must neither stick in the groove nor be too loose ; rings which lose their flexibility or are worn should be replaced. Seizing up marks on piston and liner should be smoothed. In case, the piston does not become gas-tight, by taking the above measures, the liner or both liner and piston have to be exchanged.

Fuel Injection System—Drain off water accumulated in fuel tank and filter. Clean fuel filter and see that liquid fuel oil flows to the suction side of the fuel pump. By fitting a stopper cone or blind flange to the delivery side of the pump and operating the plunger, one can feel when oil is leaking from the suction valve, the regulating or overflow valve or along the plunger. The clearance between fuel cam and roller should be as small as possible.

Fuel spray valves should be examined by taking them out of the engine head re-connecting the fuel pipe and acting the pump. A fine whitish and torn spray should come out of each nozzle hole when acting the pump with a jerk. The nozzle must not trickle after the pump stroke ; leaky nozzles cause quick carbonization and overheating of the pistons and soon get choked.

Governor—When the engine is stopped, the governor must allow the fuel pump to discharge the maximum quantity of fuel required for full or overload. In the counter position of the governor collar the quantity of fuel injected must be less than what is required for no load running, because otherwise the engine may start racing.

Timing—Turning the engine into the lower dead centre between exhaust and suction stroke, the exhaust roller must just leave the cam and the inlet valve roller just be caught by the cam. The roller clearances should not be more than 1/32″ and not less than 1/64″. The advance position of the fuel cam must be varied according to the inflammability of the fuel oil.

The cam should be as much advanced as possible without causing knocking of the engine.

STARTING—Starting the engine without having examined fuel level, cooling system and lubricating oil level and without testing lubricator, fuel pump and spray valve mean probable waste of energy, air and time. If the receiver is run out of compressed air, re-fill the air contained with carbonic acid.

RUNNING—During the running of the engine, the cooling water and lubricating oil supply must not be interrupted. Only a clean-kept and well-lubricated engine, the main parts of which are frequently tested, will run smoothly and free from trouble.

Operation of Filtration Plants—Filtration plants in Bengal are run on the basis of monthly analysis made by the Public Health Laboratory. The analysis is available after a week or ten days from the date of collection of sample, when the condition of the filter is usually entirely different. Consequently, the reports of these analysis are hardly of any use either in the management of filters or in the prevention of distribution of inferior water to the consumers. They simply serve the purpose of propaganda work, and may be of some academical value. In other countries (here also in larger cities like Calcutta, Bombay, Madras &c.), water is tested everyday and is not delivered to the consumers before it is found to be satisfactory.

The most unfortunate part of the affair is that the municipalities are made to pay for the analysis, although they are hardly of any use to them. We think, the best arrangement would have been to give a training in water analysis to the water-works superintendents, so that they can analyse water every day before it is delivered to the town. The district or city health officer can occasionally check their work.

Mechanical Filters—Working of filters in Bengal is not scientific, and consequently, the results obtained are not as satisfactory as they could be. They are worked by superin-

tendents, who do not possess the elementary knowledge of precipitation or filtration.

It is extravagant and highly improper to put up a rapid filter plant at a considerable cost, and then leave it in charge of a superintendent who never received any training in the art and science of coagulation or filtration. A rapid sand filter can only be relied upon when it is under scientific control, the employment of a technically trained and efficient superintendent being therefore imperative. Even under unscientific and inexperienced staff, the filters have not often given bacteriologically bad results. The essence of operation of a mechanical filter is the proper dosing of coagulant, which should be so proportioned to the quantity of water treated that the requisite standard of purification is obtained without waste of chemicals or permitting them to enter into the distribution system in free state. The right proportion of chemical can only be ascertained by the analysis of raw water and filtered water.

At the commencement of operation, it is always desirable to test the alkalinity and turbidity and pH value of raw water and the bacteriological condition of the filtered water and vary the amount of coagulant accordingly to give the best result with each quality of water. If these tests are made for one complete year, then it will be easy to prepare a table to show the amount of coagulant of particular specification necessary for waters of varying alkalinity and turbidity. In wet weather it may be necessary to test the raw water more frequently, as its condition then varies rapidly. When the raw water is of uniform quality throughout the year, the management of mechanical filters becomes comparatively easy, as the amount of chemical to be added can be determined once for all.

For efficient coagulation of water, it is essential to keep pH value of the water within the range necessary for good floculation, and also that the mixing of coagulant must be done for sufficiently long period, and at the right violence of agitation for producing good flocs. The modern mixing chambers

are being designed to produce greater violence of agitation in the first part of the basin than in the end. This arrangement, it is reported, has produced better flocs and reduced considerably the cost of chemicals.

A ready method of judging the quality of coagulation is by taking a sample of water from the end of the mixing chamber in a glass and examining it by holding it between the eyes and a window. It should show well formed flocs clearly separated from each other, and the liquid between them should look perfectly clear. If a highly turbid water is efficiently coagulated, the 'flocs' should settle to the bottom in 5 or 10 minutes' time. Usually, it will settle to such clearness that the particles of floculated matter remaining in suspension may be observed throughout the liquid when one litre beaker is used. If the water is so cloudy that after 10 minutes' settlement, the particles of flocs furthest from the eye cannot be detected, or are only fairly visible, the water very likely will not filter clear.

Another method of predicting the quality of effluent is the observation of water in the settling basins. The water will filter clear, if clear streaks of water form at the surface and the coagulated flocs after moving a short distance appear to drop underneath the clear surface stratum.

Besides the addition of coagulant, due attention should also be paid to proper and timely washing of the filters. The filtration head should not exceed more than 10 ft. The application of wash water to be gradual to the full pressure, and its rate must not be so high as to cause loss of sand. The filter is to be washed in such a manner that a portion of the coagulated floc is left over to form the basis for new filtering matte. Periodic examination of the level of the sand and its physical condition is necessary with a view to see if there has been any reduction in the depth of filtering medium, or if there are any mud balls or cracks forming in the filter.

Filter appertenances should be carefully looked after, and maintained in good condition, as the successful operation of these filters depends on the efficient working of such equipments.

In spite of proper and frequent washing of these filters, the sand grains often after sometime get coated with a hardened jelly-like substance, and thereby become larger in diameter than what they were before. This increase in diameter of sand grains reduces the normal efficiency of filters. The hardened jelly can be removed by soaking the medium with a strong solution of sodium hydroxide for 24 hours, and then washing out the liberated materials by prolonged washing before the filters are brought into operation.

Washing Equipment of Mechanical Filters—The capacity of wash water tank in case of hydraulic wash should be twice the capacity required for normally washing the filter for 5 or 6 minutes at the maximum rate without refilling the tank. The lowest draw off level of the tank must be about 40 ft. above the top water level of the filters. The rate of washing should be such that the water in the filter during washing rises 12 inches to 24 inches per minute. In case of air-wash, air is generally applied at the rate of 2 to 3 c.ft. of free air per minute per square foot of filter surface for a period of about 5 minutes and at a pressure slightly higher than the depth of water over the filter outlet. The wash air storage should be of double the capacity required for single washing as in the case of wash water tank.

The charging equipment of these tanks should be in duplicate, so that one can be used when the other is under repairs.

Slow Sand Filters—In mechanical filters, as stated before, the filtering matte is formed rapidly and artificially, whereas in slow sand filters, it is built up slowly in a natural way. Slow sand filters are suitable, especially for waters of low turbidity and color, and they work more uniformly day to day without constant technical supervision, whereas mechanical filters without such supervision do not give uniform results. Each of

these two kinds of filters has its own sphere of usefulness, and should be found to give best results when used in proper place. The cost of construction, availability and cost of filtering materials (especially fine sands) and also of land, and the character of raw water are some of the factors which determine the choice. Rapid sand filters do not give as high bacterial efficiency as slow sand filters.

The operation of this type of filters includes regulation of rate of filtration, scraping of the filter bed when clogged, removing the scraped sand from surface, raking the scraped surface, running the filters to waste after scraping, washing and replacing the sand.

Analysis of raw and filtered water should be made daily, and the rate of filtration should be adjusted accordingly. If the filtered water is of low standard of purity, the rate should be reduced and faults in filter bed detected.

The cleaning is done usually in Bengal by means of shovels with which ¼ inch to 1 inch is removed at a time. When the bed has become considerably clogged to a larger depth, it may be worked over with a garden fork and levelled off with a rake. The sand thus removed is washed or thrown away, whichever is found cheapest. After scraping, if it is found that the depth of fine sand has gone below the minimum depth for which the filter is designed, the level of fine sand is restored by the addition of new or washed fine sand. All replenishing of fine sand or other filtering medium should be done in summer, so that there may not be occasion of throwing the filter out of action in the rains when their services are most needed.

The optimum period of working of slow sand filters varies according to the character of water and season ; in worm season *algae* grow very rapidly and often shorten the effective life of the filters. They should be carefully and frequently removed with suitable instruments, if required.

In Appendix VIII, the extracts from the rules for the management of waterworks in Bengal is given, with standard forms required to be maintained by the superintendents.

Distribution System—The proper maintenance of distribution pipes comprises detection and repair of leaks, regulation of pressure to meet the requirements of the different parts of the town and the removal of deposits and extraneous growth within the mains.

The pipes in this country are laid about 3′-6″ below ground ; any leaks in pipes laid to such depth, especially in cleyey and loamy soil, usually appear at the surface above them, if the pipes are not under paved streets. In sandy soil, however, the leaks are very difficult to detect and require careful examination of the ground and perhaps opening of a long length of pipe. When leaks occur in pipes under pavement or tarmacadam road, they frequently follow the pipe discharge into an underground sewer or drain and remain undetected for years. It is by careful metering only, the amount of wastage through leakage can be estimated and a proper and watertight condition of mains can be maintained. Leaks should be repaired as soon as they are detected, whether caulking a lead-joint or replacing a portion of the cracked pipe with an extra collar joint. Among the several methods for the detection of leaks in distribution of pipes, the followings are commonly adopted :—

(*a*) direct observation, (*b*) use of water stethoscope, etc., and (*c*) by the construction of a hydraulic gradient line.

The first can hardly be called a method and is only practicable in places, where the soil under which the pipe is laid is such that leaks appear on the surface.

The second method of localization is by stethoscope, aquaphone, dectaphone, sonoscope etc. Of these, stethoscope and aquaphone shewn in Figs. 141 and 142 are essentially acoustic instruments, and leaks are located by following their sound. The end of the sounding rod is placed over the pipe, and by listening with the ear against the rod, the sound of leak may be heard. In aquaphone, dectaphone and sonoscope,

suitable arrangements are made to magnify the sound as in the case of telephone-receiver, but these instruments are generally with electrical connection. If the pressure is determined at

Fig. 141—Stethoscope.

several points in a pipe line when town supply is stopped and pressure is maintained, the hydraulic gradient can be plotted. If this is plotted through the points found by measurement, the kinks in the hydraulic gradient line will show the approximate position of the leak because of the change of direction of the gradient which should otherwise be uniform.

An inspection of sluice valves, washout and air valves and stop-cocks of standposts should be made periodically to see if they are in working order and are adjusted to proper opening. Circulation is often interfered with owing to the unnecessary closing of sluice valves producing dead ends, and also the unnecessary closing of valve interferes with supply of a section of the town. All the valves in the system should close clockwise, which should be marked with an arrow on the top.

Fig. 142—Aquaphone.

Cleaning of pipes are generally done by opening washout valves or hydrants and allowing the water to flush out the deposit within the pipe. The work should be done in sections of the mains one after another, so that only a small portion of the consumer is affected at a time. In case, if ordinary flushing does not answer the purpose, wooden balls may be inserted into the main through a hatch-box or other arrangements, and forced along by the pressure of water to the end of

the pipe to be cleaned. If carefully performed, this seems to be the cheapest and fairly efficient method.

In conclusion, the writer wants to point out the importance of keeping complete records and plans in the office of the superintendent of every section of the works, their administration, operation and maintenance. A set of completion plan of the whole works shewing the size, length and exact location of every pipe, house-connections, standposts, sluice valves, washout pipes should be kept in an up-to-date condition. The day-book of the pumping station should show the number and time each man worked, the number of hours the different pump and their accessories worked, the amount and calorific value of fuel consumed, the revolutions at which they worked, the quantity and quality of water pumped, its suction and delivery head, and in the case of filter, the filtration head, rate of filtration, the time and duration of washing, the quantity and quality of water filtered, and quantity and quality of coagulant used, the turbidity and alkalinity of raw water etc.

General forms annexed to the waterworks management rules may be employed, or they can be modified to suit the local conditions.

APPENDICES

APPENDIX I.

WATER ANALYSIS.

(By K. C. Banerjee, Junior.)

The following apparatus and chemicals are required for carrying out an analysis of water for sanitary purposes:—

I. Apparatus.

Chemical Balance—to read correctly up to four places of decimals with necessary weights.

Steam Oven.

Gas burners or Spirit Stoves.

Distilling Flask.

Leibig's Condenser.

Porcelain basins and dishes.

Beakers.

Conical Flasks.

Measuring cylinders—250, 100, 25, 10 c.cs.

Nessler Glasses.

Test Tubes.

Droppers.

Burette with stand—100, 50, c.cs.

Pipettes—50,25,10 c.cs.

Funnels.

Centrifuse.

Jena Flasks.

Kjeldahl Flasks.

Desiccator with calcium chloride.

Perfectly white porcelain plate.

Filter Papers.

Litmus papers—Red and Blue.

II. Chemicals—Standard Solutions.

Potassium Permanganate—1 c.c. $\equiv$ 0·1 mg. of Available Oxygen.

Sodium Thio-sulphate—Deci-normal.

Silver Nitrate—1 c.c. $\equiv$ 1 mg. of Chlorine in Chloride.

Ammonium Chloride—1 c.c. ≡ 0·01 mg. of N.
Sodium Nitrite—1 c.c. ≡ 0·001 mg. N.
Ferric Chloride—1 c.c. ≡ 0·1 mg. of Iron.
Naphthylamine.
Sulphanalic Acid.
Nessler Solution.
Sodium Carbonate.
Sulphuric Acid—Strength 1 in 3.
Potaasium Iodide.
Starch solution—1 in 100.
Potassium Chromate—1 in 10.
Alkaline Permanganate.
Oxalic Acid—2% solution.
Manganese Chloride—33% solution.
Mixed solution of Potassium Iodide and Hydroxide.
Soap solution.
Calcium Chloride.
Deci-normal Sulphuric Acid.
Deci-normal Sodium Carbonate.
Ferric Chloride.
Sodium Acetate.
Barium Chloride.
Ammonium Molybdate.
Potassium Ferocyanide.
Methylene Blue.
Methyl Red.
Phenol-phthalein.
Methyl-Orange.
Distilled Water.

III. **Items of Analysis.**

For sanitary purposes, the following analysis are usually made of a sample of water:—

Solids—

(i) Total Solids.
(ii) Suspended Solids.
(iii) Mineral and Organic matters in Suspended Solids.

Free and Saline Ammonia.
Albuminoid Ammonia.
Chlorides.
Sulphates.
Phosphates.
Nitrites.
Nitrates.
Nitrogen.
Oxygen Absorption—
(i) 3 minutes' test.
(ii) 4 hours' test.
(iii) For matters in Colloidal and True solutions.
Dissolved Oxygen.
Total Hardness.
Permanent Hardness.
Temporary Hardness.
Iron.
Sodium Carbonate.
Acidity and Alkalinity.
Free Alkalinity.
Hydrogen—lon Concentration.
Free Chlorine.

IV. Preparation of Standard Solutions.

Distilled water should always be used in preparing the Standard solutions and also in carrying out the analysis.

Potassium Permanganate Solution—Dissolve fully 0·395 gram of Potassium Permangante crystals in distilled water, and make it up to 1 litre. Each c.c. of the solution contains 0·1 mg. of Oxygen available for oxidation.

Deci-Normal Sodium Thio-Sulphate Solution—Dissolve 24-823 grams of pure crystallised salt of Sodium Thio-sulphate fully in distilled water and make it up to a litre. (This solution undergoes chemical change when left exposed to sunlight, for which 2 grams of Potassium Bicarbonate are added to each litre of solution to extend its permanancy and should always be kept in coloured bottles).

Silver Nitrate Solution—This solution is prepared by dissolving perfectly 4·797 grams of crushed crystals of pure Silver Nitrate (dried in an air oven at 105° C.) in a litre of distilled water. Each c.c. of such solution is equivalent to 1 mg. of Chlorine.

Ammonium Chloride Solution—Dissolve 3·8143 grams of pure dry Ammonium Chloride salt in a litre of distilled water. Take 10 c.cs. of such solution, and dilute it to a litre with ammonia-free distilled water. Each c.c. of the second solution contains 0·01 mg. of Nitrogen.

Sodium Nitrite Solution—To a cold solution of commercial Sodium or Potassium Nitrite, add solution of Silver Nitrate as long as precipitate appears. Decant the clear liquid above and thoroughly wash the precipitate with cold water, then dissolve it in boiling water. Concentrate and crystalise the Silver Nitrite from the hot solution. Dry it in the dark at ordinary room temperature (vacum is better) and keep it in black bottle.

Weigh out 0·22 gram of the crystallised salt of Silver Nitrite just prepared and dissolve it in *hot* distilled water, and add slight excess of Sodium Chloride solution. Cool it and dilute it to a litre with distilled water. Allow the precipitated Silver Chloride to settle, decant 5 c.cs. of the clear solution and dilute the same to a litre. The second solution thus prepared is the required standard Sodium Nitrite solution and each c.c. of this contains nitrite equivalent to 0·001 mg. of Nitrogen.

Ferric Chloride Solution—(*For Estimation of Iron*). Take pure Iron Wire weighing exactly 0·1 mg. and add to it about 5 c.cs. of dilute Hydrochloric Acid and heat until the whole thing dissolves fully, and then add to it a little of Potassium Chlorate and heat till there is any smell of chlorine. Allow it to cool. Make it up with distilled water to a litre. Each c.c. of this solution is equivalent to 0·1 mg. of Iron.

α—Naphthylamine—(*Naphthylamine Hydro-Chloride*)—5 grams of the pure salt is dissolved in a litre of distilled water

containing 8 c.cs. of concentrated Hydrochloric Acid and this is used as a standard solution.

Sulphanalic Acid—Dissolve 8 grams of the acid in a litre of distilled water containing 50 c.cs. of concentrated Hydrochloric Acid.

Nessler Solution—Dissolve 35 grams of Potassium Iodide in about 200 c.cs. of *ammonia-free* distilled water. Add to it saturated solution of Mercuric Chloride till a faint show of excess is indicated. Now add to it 160 grams of solid Potassium Hydroxide. Dilute it to a litre with distilled water. Finally add strong solution of Mercuric Chloride little by litlle until red Mercuric Iodide just begins to precipitate. Allow the solution to settle and decant off the clear solution and filter. The finished solution should have a *pale straw colour.* Sometimes it takes 2 or 3 days to prepare a good solution. It improves with age. The finished solution should be kept for a long time when it is fit for use.

Sodium Carbonate Solution—It is a deci-normal solution and contains 5·3105 grams of the salt per litre.

About 9 grams of the bi-carbonate are spread over in a thin layer on a porcelain dish, and are weighed very carefully with the dish. It is then heated over a Bunsen flame or stove to dull redness for about 20 minutes. Care should be taken that the salts do not fuse while heating, as in that case some of the carbonate will decompose. Allow the dish with its contents to cool in a desiccator, and after it has cooled down, it is again weighed carefully. The process of heating, cooling and weighing is thus repeated several times till a constant weight is arrived at.

After the constant weight is obtained, weigh out accurately 5·3105 grams of it and dissolve it in ammonia and carbon-dioxide-free doubly distilled water and make it up to a litre.

Sulphuric Acid—To 200 c.c.s of distilled water, add little by little 100 c.cs of concentrated Sulphuric Acid in a porcelain pot or in a Jena flask partially immerged under water in a sink, stirring the quantity all the while. Allow it to cool, when the solution thus obtained is ready for use.

Potassium Iodide Solution—(10% Solution).— Dissolve 10 grams of the crystallised salt in 100 c.c.s of distilled water.

Starch Solution—About a gram of powdered starch is made into a cream with a small quantity of distilled water. 100 c.cs. of boiling distilled water is then added to it and boiling is continued for few minutes. Allow it to cool, if a clear solution be not obtained, filter it off to get a clear solution for use.

Potassium Chromate Solution—Dissolve perfectly 2 grams of the pure crystallised salt in 100 c.cs. of distilled water. should the solution be not perfectly free from chloride, add a little of silver nitrate solution till a red precipitate begins to form. Allow the precipitate to ssttle down, and decant off the clear solution for use.

Alkaline Permanganate Solution—In preparing this solution, 200 grams of solid Potassium Hydroxide and 8 grams of crystallised Potassium Permanganate ($KMnO_4$) are dissolved in 1,250 c.cs. of distilled water. This solution is boiled down to 1,000 c.cs. in a porcelain pot, when it is fit for use.

Oxalic Acid Solution—Dissolve 2 grams of pure crystallised Oxalic Acid in 100 c.cs. of distilled water which gives a 2% solution. The solution becomes weaker with age and should restandardised each time when using it.

Manganese Chloride Solution—Dissolve perfectly in 100 c.cs. of distilled water 33 grams of the salt which gives a 33% solution.

Mixed Potassium Iodide and Hydroxide Solution—In 100 c.cs. of distilled water, dissolve 70 grams of solid Potassium Hydroxide and 10 grams of Potassium Iodide crystals to get the required solution.

Soap Solution (1 c.c. $\equiv$ 2·5 mg. of $CaCO_3$) —100 c.cs. of saturated solution of Castile soap in Methylated spirit, after repeated filtration, are mixed with about 320 c.cs of filtered spirit, and the resulting solution is approximately of the requisite strength. This solution is then standardised against

a standard solution of Calcium Chloride or Barium Nitrate in the following way :—Prepare a strong Soap Solution by dissolving Castile Soap in a mixture of equal volumes of methylated spirit and distilled water. Filter the solution. This should now be standardised against a solution of Calcium Chloride prepared from pure Calcium Carbonate and Hydrochloric Acid. But as pure Calcium Carbonate is not generally available, Barium Nitrate, which is soluble in water, is taken as its substitute.

261 grams of $Ba(NO_3)_2 \equiv 100$ grams of $CaCo_3$.

$\therefore$ 0·261 mg. of $Ba(NO_3)_2 \equiv 0{\cdot}1$ mg. of $CaCO_3$.

0·261 mg. of $Ba(NO_3)_2$ is dissolved in distilled water and made up to 1 litre. 50 c.cs. of this solution are taken in a stoppered phial and titrated against the soap solution, so that 2·2 c.cs. of it added to 50 c.cs. of Barium Nitrate solution will give a lather lasting five minutes. 0·2 c.cs. is taken in forming a lather with 50 c.cs. of the distilled water. Now, to effect this, the following procedure is adopted :—

The soap solution is taken in a burette and 50 c.cs. of the standard Barium Nitrate solution in a 150 c.cs. stoppered bottle. Now, add soap solution a little at a time to the Barium Nitrate solution, and after each addition insert the stopper and shake. Continue this, and towards the end add drop by drop until a permanent lather is formed.

Suppose 1·8 c.cs. of the soap solution was used up and, in that case the soap solution was too strong ; for every 1.8 c.cs. —0·4 c.c. of a mixture of methylated spirit and distilled water would be required to be added. If originally 180 c.cs. of soap solution were prepared 40 c.cs. of spirit and water are to be added and a fresh 50 c.cs. of Barium Nitrate solution are titrated with this solution. This process is continued until 2·2 c.cs. of the soap solution become equivalent to 50 c.cs. of Barium Nitrate solution.

Now 50 c.s. of Barium Nitrate $\equiv$ 5 mg. of $CaCO_3$.

But 50 c.cs. of barium Nitrate $\equiv$ 2·2 c.cs. of Soap Solution

$\equiv$ 2 c.cs. of soap solution.

(As 0·2 c.c. is due to distilled water)

∴ 2 c.cs. of the soap solution = 5 mg. of CaCO·.

∴ 1 c.c. of the soap solution = 2·5 mg. of $CaCO_3$.

Calcium Chloride Solution—Weigh out accurately 0·2 gram of pure Ice-land spar and dissolve it in dilute Hydrochloric acid, taking care to keep the porcelain pot covered all the while to prevent loss by spirting. Evaporate the solution to dryness on an water-bath, add distilled water and evaporate it again to dryness as before and repeat the process at least twice to remove all free Hydrochloric acid. Now, dissolve the residue of the neutral salt in distilled water and make it up to a litre.

Deci-Normal Sulpuric Acid—This is prepared by diluting 100 c.cs. of the normal acid to a litre with distilled water.

Ferric Chloride Solution—(*For estimation of Dissolved Oxygen*).—This solution is prepared by perfectly dissolving 5 grams of the crystallised salt in 100 c.cs. of distilled water.

Sodium Acetate Solution—This solution is prepared by fully dissolving 5 grams of Sodium Acetate in 100 c.cs. of distilled water.

Barium Chloride Solution—A saturated solution of this salt is always used.

Methylene-Blue Solution—Dissolve 0·5 gram of the commercial salt per litre of distilled water which is used as an indicator.

Methyl-Red Solution—This solution is prepared by dissolving 0·2 gram of the methyl-red in a litre of *hot* distilled water ; the solution is then allowed to cool and filtered off, if necessary.

It changes in colour from pale yellow to violet red which is very distinct and renders it useful for titration of weak bases with strong acids.

Phenol-Phthalein Solution—It is prepared by dissolving 0·2 gram of pure phenol-phthalein in 60 c.cs. of rectified spirit along with 40 c.cs. of distilled water and filtered, if required.

It gives red colouration with alkali and the colour is discharged with excess of acid.

Methyl Orange Solution—Dissolve 1 gram of Methyl Orange in a small quantity of methylated spirit, and make it up to a litre with methylated spirit and dilute it with its own volume of distilled water.

This solution does not undergo any change or decomposition with age.

V. Procedure of Analysis.

SOLIDS.

Apparatus.

Chemical Balance with weights.
Measuring Cylinder.
Porcelain Basin.
Funnels.
Filter Papers.
Steam-Oven.
Desiccator with chemicals (Calcium Chloride).
Centrifuse.

(i) Total Solids.

Procedure—Weigh a clean, dry porcelain basin very accurately, and take exactly 100 c.cs. of the sample of water and evaporate it on an water-bath and dry it perfectly in a steam oven. Allow it to cool in a desiccator containing Calcium Chloride. Weigh it again. The difference in weight (of the crucible and the second weight) gives the weight of total solids present in 100 c.cs. of the sample, and from this, parts per 100,000 is the calculated.

(ii) Suspended Solids.

Procedure—Filter a portion of the sample to separate the suspended solids and take exactly 100 c.cs. of the filtrate into a weighed basin and evaporate it on an water-bath and dry it perfectly in a steam-oven and cool it in a desiccator. Weigh

the basin with its contents. Let the weight of the basin alone be 'x' grams and the weight of the basin with its contents be 'y' grams.

Weight of dissolved solids in 100 c.cs. of the sample—

$(y-x)$ grams . . (1)

but Total solids = Dissolved solids + Suspended solids.

Suspended solids = Total solids − Dissolved solids . . (2)

Quantity of total solids present in 100 c.cs. of the sample is determined before and from (1) the quantity of dissolved solid is known, and hence the amount of suspended solids present in the sample is calculated from equation (2).

(iii) Mineral and Organic Matters in Suspended Solids.

Procedure—First centrifuse a portion of the sample, whereby all the suspended matters are deposited at the bottom with a clearn liquid above. Decant off the clear liquid and take the deposit on a crucible and evaporate it on an water-bath and dry it in a steam-oven. Take a known weight (say 0·5 to 1·00 gram) of the dried solid on an weighed crucible and ignite it fully on a flame. The colour becomes brownish or whitish other than black on perfect ignition. Allow it to cool in a desiccator. The difference in weight between the crucible plus the weight of the solid taken and the weight of the crucible with its contents after ignition gives the weight of the organic matters present in the solid, *i.e.* in the sample.

On perfect ignition all the organic matters present in the solids burn away, the ash that remains in the crucible is nothing but mineral matters.

If 'x' grams be the weight of the crucible, 'y' grams be the weight of the crucible with the solid before ignition, and 'z' grams be the weight of the crucible with ash, then

(1) Weight of Organic substance = $(y-z)$ grams.

(2) Weight of ash = $(z-x)$ grams.

From (1) and (2) the percentage or parts per 100,000 of the mineral or organic matters present in the sample may be calculated.

Free and Saline Ammonia.

Apparatus.	*Chemicals.*
Distillation Flask.	Pure Anhydrous Sodium Carbonate.
Leibig's Condenser.	Nessler Solution.
Stove or Bunsen Burner.	Ammonium Chloride Solution (1 c.c.$\equiv$0·1 mg.N).
Nessler Glasses.	Distilled water.
Measuring Cylinders.	
White Porcelain Plate.	
Pumice Stones.	

Procedure.

The apparatus are all first made ammonia-free.

Take 250 c.cs. of the sample in a distillation flask, and to it add a little of pure anhydrous Sodium Carbonate and put in a few pieces of pumice stones to prevent bumping while boling. Connect the flask tightly to a Leibig's condenser and start distillation. Distil over 100 c.cs. into a Nessler glass with two marks—one at 50 c.cs., and the other at 100 c.cs. Of the 100 c.cs. distillate thus obtained, take 50 c.cs. into a 50 c.cs. Nessler glass and add 2 c.cs. Nessler solution. Allow it to stand for 5 minutes for the colour to develop.

Take 0·5, 1·0 and 1·5 c.cs. of Standard Ammonium Chloride solution (strength 1 c.c.$\equiv$0·01 mg. N.) in three separate 100 c.cs. Nessler glasses and dilute the volume up to 50 c.cs. with ammonia free distilled water in each case, and then add 2 c.cs. of Nessler solution to each and note the colour developed in each.

Pour out little by little in a measuring cylinder the 50 c.cs. distillate treated with Nessler reagent before to match in colour with any of the standards prepared just before by placing these on a perfectly white plate. Let it match with a colour which is between 1 and 1·5, say 1·2.

100 c.cs. of the distillate are collected because, this quantity is sufficuient in as much as there remains practically no free and saline ammonia after 100 c.cs. are distilled off.

Calculation.

50 c.cs. of the distillate $\equiv 1{\cdot}20$ c.cs. of standard NH_4Cl solution
$\equiv 1{\cdot}20 \times 0{\cdot}01$ mg. of N,

$\therefore$ 100 c.cs. of the distillate $\equiv 1{\cdot}20 \times 0{\cdot}2 \times 0{\cdot}01$ mg. of N,
i.e. 250 c.cs. of the sample $\equiv 1{\cdot}20 \times 2 \times 0{\cdot}01$ mg. of N,
$\therefore$ in $250 \times 4 \times 100$ i.e. in parts per 100,000
$= 1{\cdot}20 \times 2 \times 0{\cdot}01 \times 4 \times 100$ mg. of N.
$= 1{\cdot}20 \times 2 \times 0{\cdot}01 \times {\cdot}4$ gr. of N.

In short, if 50 c.cs. of the 100 c.cs. distillate $= y$ (say) then Free Ammoniacal Nitrogen in parts per 100,000 $= y \times 0{\cdot}008$.

Albuminoid Ammonia.

Apparatus.

Distillation Flask.
Leibig's Condenser.
Stove or Bunsen Burner.
Nessler Glasses.
Measuring Cylinders.
White Porcelain Plate.
Pumice Stones.

Chemicals.

Pure Anhydrous Sodium Carbonate.
Nessler Solution.
Ammonium Chloride Solution. (1 c.c. $\equiv$ 0·01 mg. of N).
Alkaline Potassium Permanganate solution.
Distilled water.

Procedure.

To the mother liquor left after the distillation of Free and Saline Ammonia, add 50 c.cs. of Alkaline Potassium Permanganate solution and 100 c.cs. of ammonia free distilled water, thus making the total volume to 250c.cs. Distil and collect 100 c.cs. and proceed exactly as before.

The calculations are exactly the same as for the Free and Saline Ammonia.

Chlorides.

Apparatus.

Measuring Cylinder.
Porcelain Basin.
Dropper.
Burette with stand.

Chemicals.

Standard Silver Nitrate Solution. (1 c.c. $\equiv$ 1 mg. of Cl.).
Potassium Chromate Solution (Indicator).

Procedure.

Take 100 c.cs. of the sample in a porcelain basin (filtering is necessary if the sample contains much suspended matter, otherwise, the original sample may be taken for analysis), and add to it a drop or two of Potassium Chromate solution, which is used as an indicator and gives a yellowish colouration to the liquid.

Titrate it with standard Silver Nitrate solution (1. c.c.$\equiv$ 1 mg. Cl). Just at the point, where colour changes from *straw yellow to faint red colouration*, stop addition of any more silver nitrate solution. Note the quantity required, which determines the end point of titration. Let the amount of Silver Nitrate required be 'x' c.cs.

Note.—If the chlorine content is high, then 25 c.cs. or 10 c.cs. are taken. If 10 or 25 c.cs. are taken, the volume is diluted to 100 c.cs. with distilled water, otherwise, it is difficult to titrate with 10 or 25 c.cs. of the sample.

Calculation.

x c.cs. of $AgNO_3 \equiv x$ mg. of Cl_2

But this x c.cs. of $AgNO_3 \equiv 100$ c.cs. of the sample taken originally,

$= x$ mg. of Cl_2

$\therefore$ in parts per 100,000 x grams.

Sulphates.

Apparatus.	*Chemicals.*
Flask.	Barium Chloride—10% solution.
Funnels.	Hydrochloric Acid.
Filter Papers.	Distilled Water.
Measuring Cylinders.	

Procedure.

Qualitative Test—100 c.cs. of the sample are taken in a flask and boiled until one third of the volume is left. It is then filtered and allowed to cool and again made up to original volume with distilled water. Of this, 50 c.cs. are kept for

estimation of permanent hardness. To the remaining 50 c.cs. add 10% Barium Chloride solution and Hydrochloric Acid, and allow it to stand for 5 minutes when Barium Sulphate will be precipitated indicating the presence of Sulphate in the sample.

Quantitative Analysis—Acidify 500 c.cs. of the sample with Hydrochloric Acid and evaporate it to about 50 c.cs. Add Barium Chloride solution in exess to it when Sulphate present in the liquid is precipitated down. Filter off the precipitate. Transfer the precipitate in a basin and dry it in a steam oven. Ignite and weigh the residue of the sulphate after ignition, which gives the weight of sulphate present in 500 c.cs. of the sample.

Then calculate parts per 100,000.

Phosphates.

Apparatus.	*Chemicals.*
Porcelain Basin.	Concentrated Nitric Acid.
Test Tubes.	Dilute Nitric Acid.
Steam Oven.	Ammonium Molybdate Solution.
Measuring Cylinders.	
Glass Rod.	
Funnel.	
Filter Papers.	

Procedure.

Qualitative Test—Add a little of concentrated Nitric Acid to 50 c.cs. of the sample and evaporate it to dryness on an water bath and dry it in a steam-oven. After it is well dried, add to it 2 to 3 c.cs. of dilute nitric acid and filter. Heat the filtrate, but not very strongly, and add 3 c.cs. Ammonium Molybdate solution and rub the sides of the vessel or tube all the time while adding the molybdate solution. The liquid turns yellow in the presence of Phosphates while a yellow precipitate indicates the presence of heavy amounts of Ammonium Phosho-molybdate.

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Nitrites.

Apparatus.	*Chemicals.*	
Nessler Glasses.	Sulphanalic Acid Solution.	For Qualitative Analysis.
Measuring Cylinder.	α-Naphthylamine Solution.	
Test Tube.	Starch Solution.	
White Porcelain Plate.	Potassium Iodide Solution.	
	Dilute Sulphuric Acid.	
	Sodium Nitrite Solution.	
	Sulphanalic Acid Solution	
	α-Naphthylamine Solution.	
	Distilled Water.	

Procedure.

(I) *Ilosvay's Test.*

Qualitative Tests—50 c.cs. of the sample are taken in a nessler glass and about 1 c.c. (equal volumes) each of Sulphanalic Acid and α-naphthylamine solutions are added to it when pink colouration develops indicating the presence of Nitrites in the sample. If no colour develops within 15 minutes, nitites may be considered as absent.

This method is very sensitive and reliable.

(II) *Starch Iodide Test*—It consists in the addition of a little of freshly boiled and cooled Starch solution and a minute quantity of freshly prepared Potassium Iodide solution to a small quantity of the sample in a test tube. Dilute Sulphuric acid is then added, when the presence of nitrite is indicated by the *immediate* appearance of *blue colour* but *if the colour develops after sometime has elapsed, it is due to Nitrate and not Nitrite.*

Quantitative Analysis—The amount of nitrite present in a sample of water is determined by comparison of colours with a

standard as was adopted in cases of Free or Saline and Albuminiod Ammonia determination.

Stanard—Take 1 c.c. of Standard Sodium Nitrite solution (1 c.c.$\equiv$0·001 mg. N) in a nessler glass and dilute it to 96 c.cs. with distilled water, then add to it equal volumes of α-Naphthylamine and Sulphanalic acid (generally 2 c.cs. of each are taken), thus making the whole to 100 c.cs. Wait for 30 minutes or so when a light pink colouration develops, which is used as a standard.

Comparison—Take 50 c.cs. of the filtered sample (quantity of water to be taken depends upon the amount of nitrite present in the sample and filtering is not necessary, if the sample does not contain much suspended solids) in a nessler glass and make it up to 96 c.cs. with distilled water, and add to it 2 c.cs. each of α-Naphthylamine and Sulphanalic acid solutions. Allow it to stand at least 15 minutes within which time a pink colour develops. Now, compare its colour with the standard prepared for the purpose and match the two colours exactly in the following manner.

Look at the bottom of the two tubes, and if the two differs in the density of colours—pour out, little at a time, the liquid (the one which has the higher colour density) in a measuring cylinder until colours of the liquids coincide exactly. The difference between 100 c.cs. (taken first) and the quantity poured out into the cylinder gives the volume required to match with the standard (prepared) and let it be 35 c.cs.

Calculation.

Let 35 c.cs. of the liquid match with 100 c.cs. of the standard.

Again 100 c.cs. of the standard$\equiv$ 1 c.c. of Sodium Nitrite Solution,

$\therefore$ 35 c.cs. of the liquid$\equiv$0·001 mg. (N).,

$\therefore$ 100 c.cs. of the liquid$\equiv\frac{0{\cdot}001 \times 100}{35}$ mg. (N),

But this $\frac{0{\cdot}001 \times 100}{35}$ mg. (N) is derived from 50 c.cs. of the sample originally taken,

∴ 100 c.cs. of the original sample

$$= \frac{0{\cdot}001 \times 100}{35} \times \frac{100}{50} \text{ mg. (N)},$$
$$= 0{\cdot}006 \text{ mg. (N)}.$$
$$= 0{\cdot}006 \text{ grams per } 100{,}000.$$

Nitrates.

Apparatus.

Stoppered Bottle.
Distillation Flask.
Leibig's Condenser.
Measuring Cylinder.
Beakers.

Chemicals.

Zinc Foils.
Dilute Sulphuric Acid.
Saturated Copper Sulphate Solution.
Acetic Acid.
Solid Sodium Carbonate.
Distilled Water.

Procedure.

Zinc-Copper Couple Method—Zinc-copper couple is prepared by keeping immersed Zinc foils in a saturated colution of Copper sulphate. The Zinc foils are first cleansed with dilute Sulphuric acid, next washed with distilled water and then kept immersed in a saturated solution of Copper sulphate for about 3 minutes when an adherent deposit of metallic copper is formed on the Zinc foils. The foils are then removed from the bath, otherwise the coating becomes powdery. The foils are then washed with ammonia-free distilled water.

Take 100 c.cs. of the sample in a stoppered bottle, and put in some Zinc-copper couple in it and add to it 2 or 3 drops of Acetic acid. Allow the whole thing to stand overnight. Next day, decant off the 100 c.cs. of water into a distillation flask and wash the bottle with ammonia-free distilled water. The washings together with the distilled water amounting to 150 c.cs. should now be added to the distillation flask thus making the volume 250 c.cs. in all. Add a little of solid Sodium Carbonate and distil over ammonia in the same manner as described under Free and Saline Ammonia. 150 c.cs. of the distillate should be collected. Of the 150 c.cs. of the distillate,

50 c.cs. are taken and 2 c.cs. of nessler reagent is added to it and proceed in the same way to determine amount of Ammonia as described before. Note the result. The amount of original Free and Saline Ammonia is to be deducted from this result which gives the amount of Nitrate present in the sample. Generally nitrites are absent in water, but where present, the amount of nitrite is also to be deducted from this result.

Calculation.

Let 50 c.cs. of the distillate $\equiv 0{\cdot}4$ c.c. of standard solution.
$\equiv 0{\cdot}4 \times 0{\cdot}01$ mg. (N),
$\therefore$ 150 c.cs. of the distillate $\equiv 0{\cdot}4 \times 0{\cdot}01 \times 3$ mg. (N),
i.e., 100 c.cs. of the sample $\equiv 0{\cdot}4 \times 0{\cdot}01 \times 3$ mg. (N),
$\therefore$ parts per 100,000 $\equiv 0{\cdot}4 \times 0{\cdot}01 \times 3$ gr. (N),
$\equiv 0{\cdot}012$ gram. (N).

Let the amount of Free and Saline Ammonia be
$= 0{\cdot}0096$ gr. (N),
$\therefore$ Nitrates present $= 0{\cdot}012 - 0{\cdot}0096$ grams.
$= 0{\cdot}0024$ parts per 100,000.

Nitrogen.

Apparatus.

Chemical Balance with necessary weight.
Porcelain Basins.
Steam Oven.
Bunsen Burner or Spirit Stove.
Kjeldahl's Flask.
Distillation Flask.
Pumice Stones.
Beakers.
Leibig's Condenser.
Funnels.
Measuring Cylinders.
Dropper.
Litmus Paper.

Chemicals.

Concentrated Sulphuric Acid.
Crystallised Copper Sulphate.
Potassium Sulphate (Solid).
Sodium Hydroxide Solution —40%.
Deci-Normal Sulphuric Acid.
Deci-Normal Sodium Carbonate Solution.
Methyl Orange Solution.
Litmus Papers.
Distilled Water.

Procedure.

Take a quantity of the sample of water under analysis after thorough shaking evaporate it to dryness on an water-bath and dry it in a steam-oven. Take a certain weight of the dried solid, say from 0·5 to 1·0 gram, (exact weight should always be taken) in a Kjelhadhl's flask and pour in 25 c.cs. of concentrated Sulphuric acid together with a small crystal of Copper Sulphate which here acts as a catalyser. Cook it for half an hour, then add 7 to 10 grams of solid Potassium Sulphate, which raises the temperature of the mixture. Again, continue the process of cooking till the mixture becomes quite clear. Allow the mixture to cool. Then transfer it completely with distilled water in a distillation flask and dilute it to 400 c.cs. with distilled water. Put in a red litmus paper into the flask to ensure that the top layer of the liquid is acidic, otherwise red litmus turns blue with alkali. Now, add 100 c.cs. of 40% Sodium Hydroxide solution very slowly and trickling down the sides of the flask and without disturbing the upper layers of the liquid. The alkali being heavy settles down at the bottom and any ammonia that evolves is dissolved in the acid layers above. Put in some pumice stones to prevent bumping when boiling. Make it sure before putting for distillation that the whole liquid in the flask is alkaline.

Take 50 c.cs. of deci-normal Sulphuric acid in a beaker and add to it a drop or two of Methyl Orange. Fit in a funnel to the second end of the Leibig's condenser by means of a rubber tubing and keep the other end of the funnel dipped in the sulphuric acid in the beaker which prevents escape of any Nitrogen. Now start distillation after shaking the liquid thoroughly. Collect the distillate in the beaker containing the sulphuric acid till there is no ammonia in the mother liquor, which is tested with litmus paper. Wash the funnel with ammonia-free distilled water into the beaker containing the distillate. Test the distillate with litmus paper for ammonia. Then titrate with standard deci-normal solution of Sodium Carbonate.

Calculation.

Let 20 c.cs. of Na_2CO_3 (N/10) are used up for back titration,

$\because$ 20 c.c.s. ,, ,, $\equiv$ 20 c.cs. of N/10 H_2SO_4,

Hence, this 20 c.cs. of N/10 H_2SO_4 must have been left in the beaker out of 50 c.cs. taken originally,

$\therefore$ 50 − 20 c.cs. = 30 c.cs. of N/10 H_2SO_4 must have been used up by ammonia evolved from 1 gram of the solid taken originally.

Again, 30 c.cs. of N/10 $H_2SO_4 \equiv$ 30 c.cs. of N/10 NH_3,

1000 c.cs. of (N) NH_3 contains 17 grams,

$\therefore$ 1 c.c. ,, ,, ,, $\frac{17}{1000}$ gram 0·017 gram,

$\therefore$ 1 c.c. N/10 ,, ,, 0·0017 gram,

$\therefore$ 30 c.cs. ,, ,, ,, $0{\cdot}0017 \times 30 = 0{\cdot}051$ gram.

Hence, 0·051 gram of NH_3 must have been evolved from 1 gram of the solid originally taken.

Again, in 17 grams of NH_3 there are 14 grams of Nitrogen (N_2),

$\therefore$ in 0·051 gram ,, ,, $\frac{0{\cdot}051 \times 14}{17}$ gram of N_2,

$\therefore$ in 100 grams ,, ,, $\frac{0{\cdot}051 \times 14 \times 100}{17}$ grams of N_2,

$= 4{\cdot}2$ grams of N_2.

Oxygen Absorption.

Apparatus.

Stoppered Conical Flask or Bottle.
Measuring Cylinders.
Burette.
Pipette.
Dropper.
Glass Rod for stirring.

Chemicals.

Standard Potassium Permanganate Solution.
Sulphuric Acid (1 : 3).
Potassium Iodide Solution (1 in 10).
Starch Solution.
Potassium Iodide Crystals.
Ferric Chloride Solution 5%.
Sodium Acetate Solution 5%.
Ferric Alum.

Procedure.

3 Minutes Test—Take 25 c.cs. of standard Potassium Permanganate solution in a conical flask or bottle (the quantity of Permanganate required depends upon the amount of organic matter present in the sample). Add to it 10 c.cs. Sulphuric Acid (strength 1 : 3). Pour 50 c.cs. of the sample into the conical flask containing the Permanganate. *Put in the stopper to the flask very tightly such that no air from outside can enter into the flask.* Allow the mixture to stand *just* for 3 minutes and add 5 c.cs. of Potassium Iodide solution (1 : 10) which stops reaction altogether liberating at the same time free Iodine. Then titrate it with standard Sodium Thio-Sulphate solution 1 c.c. $\equiv$ 0·1 mg. O_2) using Starch solution as an indicator.

In titrating with Thio-sulphate solution kept prepared long before, a blank titration of the permanganate and sulphuric acid with thio-sulphate should always be made first each time and the factor should be calculated and taken into account in calculating Oxygen Absorption as the strength of Thio-Sulphate decreases with age and also in presence of light.

Blank Titration—Let 16 c.cs. of Thio-sulphate solution are required for 25 c.cs. of Potassium Permanganate,

$$\text{Factor} = \tfrac{25}{16} = 1{\cdot}60.$$

Calculation.

Let 15 c.cs. of Thio-sulphate are required for back titration,

$\therefore$ permanganate required $25 - 15 = 10$ c.cs.,

This 10 c.cs. of Permanganate

$\equiv 10 \times 1{\cdot}6$ c.cs. of Thio-Sulphate,

$\equiv 16$ c.cs. of Thio-Sulphate,

$\equiv 16 \times 0{\cdot}1$ mg. O_2,

$\equiv 1{\cdot}60$ mg. O_2.

Again, this 1·60 mg. O_2 have come from 50 c.cs. of the sample,

$$\therefore \text{ 100 c.cs. of the sample} \equiv \frac{1{\cdot}60 \times 100}{50} \text{ mg. } O_2.$$

$$= 3{\cdot}20 \text{ mg. } O_2.$$

$\therefore$ parts per 100,000 = 3·20 grams O_2.

4 Hours Test (Tidy's Process)—Take 250 c.cs. of the sample in a 500 c.cs. stoppered bottle. To this, add 10 c.cs. of Sulphuric Acid (1 : 3) and 10 c.cs. of standard Potassium Permanganate solution (1 c.c.$\equiv$0·1 mg. of available O_2). Put in the stopper very tightly so that any air can not enter into the bottle which will vitiate the result to a considerable extent. The mixture is then allowed to stand exactly for 4 hours and *just* at the end of 4 hours a crystal of Potassium Iodide is added. Then, it is titrated with Sodium Thio-sulphate solution (the Thio-sulphate solution is prepared by dissolving 1 or 2 grams of Sodium Thio-sulphate in a litre of distilled water and in this case no standard solution of Thio-sulphate is required to be kept ready) using Starch solution as an indicator which imparts a blue colouration due to the liberation of free Iodine which disappears fully just at the point of complete titration.

In this case too, a blank titration should be made to determine the strength of the Thio-sulphate.

In cases, where water is suspected to have much organic polution, 10 c.cs. of Permanganate and 10 c.cs. of Sulphuric acid are first added to 250 c.cs. of water and it should be observed whether pink colour of the permanganate disappears or not, if it actually disappears or become very faint, another 10 c.cs. of Permanganate should be added and in the like manner requisite quantity of Permanganate is to be added till the colour of the Permanganate is permanent.

Calculation.

Suppose the blank of 10 c.cs. of Permanganate require 15 c.cs. of Thio-sulphate solution, and 250 c.cs. of water and 10 c.cs. of Permanganate require 10 c.cs. of Thio-sulphate solution.

Blank.

(15 − 10) or 5 c.cs. of Thio-sulphate are used up by organic matter,

Now, 15 c.cs. of Thio-sulphate $\equiv$10 c.cs. of Permanganate,
$\equiv 10 \times 0·1$ mg. of available O_2,

$\therefore$ 1 c.c. of Thio-sulphate $\equiv \frac{10 \times 0\cdot 1}{15}$,, ,, ,, ,,

$\therefore$ 5 c.cs. ,, ,, $\equiv \frac{5 \times 0\cdot 1 \times 10}{15}$ mg. of available O_2.

$\therefore$ 250 c.cs. of the sample absorbed $\frac{5 \times 1}{15}$ mg. of O_2,

$\therefore$ 100 c.cs. ,, ,, ,, $\frac{5 \times 100}{15 \times 250}$ mg. of O_2,

$\therefore$ 100,000 parts ,, ,, $\frac{5 \times 0\cdot 4}{15}$ grams of O_2.

In short, the difference in c.cs. of Thio-sulphate required for the blank test and 250 c.cs. of the sample together with 10 c.cs. of Permanganate and Sulphuric acid divided by the c.cs. of Thio-sulphate required for the blank test and the whole multiplied by 0·4 gives parts of available Oxygen per 100,000.

If the Thio-sulphate solution is kept prepared for ready use, then the procedure and calculations given for 3 minutes test should be adopted, otherwise, method of analysis and calculations as shewn in 4 hours' test should be done.

(iii) Oxygen Absorption for Matters in Collodial and True Solution.

Procedure.

(I) First obtain Oxygen absorption of the sample after shaking it thoroughly, which gives Oxygen absorption for total oxidisable matters present in the sample i.e., for the following—

(1) Matters in suspension.
(2) Matters in Colloidal solution.
(3) Matters in True solution.

(II) Filter off a measured quantity of the sample thus separating all the suspended solids and leaving only matters in Colloidal and True solutions. Repeat the same procedure as before and get the Oxygen absorption in this case.

(III) Separate the Colloidal matters in solution from the filtrate by treating it with 2 c.cs. of 5% Ferric Chloride and 2 c.cs. of 5% Sodium Acetate solutions and boil the mixture for 2 minutes or more if necessary. Cool and filter. If it be found that the Colloidal matters do not separate by this treat-

ment, then treat the original filtrate with Ferric Alum, which completely separates all colloidal matters in solution. Repeat the method of analysis as is adopted before and obtain the Oxygen absorption for this which gives the Oxygen consumed by matters in Colloidal solution only,

The preceding method

(I) gives Oxygen absorption for all organic matters in the sample ;

(II) gives Oxygen absorption for matters in Colloidal and True solution only ;

(III) gives Oxygen absorption for matters in True solution only.

Difference between (I) and (II) gives Oxygen absorption for matters in suspension.

Difference between (II) and (III) gives Oxygen absorption for matters in Colloidal solution.

The calculation is exactly the same as shewn before and the results are expressed in parts per 100,000.

Dissolved Oxygen.

Apparatus.

Stoppered Phials.
Pipettes.
Burette.
Dropper.
Measuring Cylinders.
Incubator.

Chemicals.

Concentrated Sulphuric Acid.
Deci-Normal Potassium Parmanganate or, Standard Permanganate solution used for Oxygen absorption determination.
Oxalic Acid Strength 2% or Potassium Oxalate Solution 2%.
Manganese Chloride Solution 33%.
Potassium Iodide and Hydro-oxide Solution (Mixture).
Concentrated Hydrochloric Acid.
Standard Sodium Thio-sulphate Solution.

Procedure.

Take a stoppered phial, whose volume is known, and fill in with the sample of water under analysis. Put in the stopper taking every precaution necessary for elmiinating air bubble remains in the phial. Take out the stopper—first add 2 c.cs. of Potassium Permanganate solution to oxidise the organic matters present in the sample. Shake the phial gently and see that the colour of the Permanganate persists after 5 minutes time. If it does not stand, add more permanganate solution till the colour is permanent after 5 minutes. After 5 minutes add 1 c.c. of 2% Oxalic Acid or Potassium Oxalate solution to decolourise the mixture—if it requires more, add. When the solution becomes colourless, add 2 c.cs. of 33% Manganese Chloride solution. Then add 1 c.c. of standard Potassium Iodide and Potassium Hydroxide solution (mixture), which gives a white spongy precipitate of Manganous Hydroxide. Allow the precipitate to settle at the bottom—then add 5 c.cs. of concentrated Hydrochloric Acid. Shake the phial very gently such that all the precipitate dissolve fully in the acid liberating free Iodine, which colours the liquid yellow. Then titrate with standard Thio-sulphate solution (1 c.c. $\equiv$ 1 mg. O_2) using Starch solution as an indicator.

The reagents should always be added to the bottom of the phial without disturbing the upper layers in the least. Care should be taken in putting the stopper to the phial each time after the addition of reagents so that no air bubble remains in it, and every precaution for this should be taken for, otherwise, it will vitiate the results considerably.

Calculation.

Let the factor for blank titration be 1·70.

Thio-sulphate required for back titration

$$= 14{\cdot}5 \text{ c.cs.,}$$
$$\equiv 14{\cdot}5 \times 1{\cdot}7 \text{ c.cs.}$$
$$= 24{\cdot}65 \text{ c.cs.}$$
$$\equiv 24{\cdot}65 \times 0{\cdot}1 \text{ mg. } O_2.$$
$$= 2{\cdot}465 \text{ mg. } O_2.$$

Let the volume of water taken be 225 c.cs.

$\therefore$ 2·465 mg. O_2 must have come from 225 c.cs. of the sample,

$\therefore$ 100 c.cs. of the sample $= \frac{2 \cdot 465 \times 100}{225}$ mg. O_2,

$= 1 \cdot 095$ mg. O_2.

$\therefore$ in parts per 100,000 will be 1·095 grams.

By Incubation—There remains some organic matter present in water that does not oxidise in 4 hours time even but require longer period and undergoes oxidation under a certain temperature, and this period is known as the *Incubation Period.* Generally, a sample is incubated under a temperature of 30°C. to 37°C. and for a period of 24 hours and 48 hours.

Take 3 phials full of water and mark them. Of the three phials, examine the contents of the first for dissolved oxygen in like manner just described and note the results. Second and the third phials are kept in the incubator under the temperature stated above. The second phial is kept for 24 hours and the third for 48 hours. After 24 hours the second phial is taken out of the incubator and the amount of dissolved oxygen present in the contents is determined in the same way as before. The difference in results of (i) and (ii) gives the amount of oxygen used up in 24 hours from which the rate of absorption is calculated. The third phial is taken out after 48 hours and the same method is adopted as before, and the rate of absorption is determined.

Hardness.

Apparatus.	*Chemicals.*
Stoppered Bottles (150 c.cs.).	Soap Solution (1 c.c. = 1 mg. $CaCO_3$).
Burette 25 or 50 c.cs.	Soap Solution 1 c.c. = 2·5 mg. $CaCO_3$).
Stop watch.	Distilled Water.
Beakers.	
Measuring Cylinders.	
Flask.	
Stove or Burner.	

Procedure.

1. *Total Hardness*—Take 50 c.cs. of the sample in a 150 c.cs. stoppered bottle. Add standard Soap solution (1 c.c. $\equiv$ 2·5 mg. $CaCO_3$) little at a time from a burette and replacing the stopper each time. Shake the bottle vigorously till a permanent lather is formed. Towards the end, add Soap solution drop by drop.

Calculation.

Suppose 50 c.cs. of the sample required 3·2 c.cs. of standard Soap solution.

Of this 3·2 c.cs., 0·2 c.cs. was required for distilled water.

∴ Nett Soap solution required	$=3{\cdot}2-0{\cdot}2=3{\cdot}00$ c.cs.
∴ 50 c.cs. of the sample	$\equiv 3$ c.cs. of Soap solution,
	$\equiv 3\times 2{\cdot}5$ mg. $CaCO_3$.
∴ 100 c.cs. ,, ,,	$\equiv 3\times 2{\cdot}5\times 2$ mg. $CaCO_3$,
	$=15$ mg. $CaCO_3$.
∴ parts per 100,000	$=15$ grams of $CaCO_3$.

(ii) *Permanent Hardness*—Take 100 c.cs. of the sample in a flask and boil it to one-third of the original volume. Filter it off and allow it to cool. Again dilute it to 100 c.cs. with distilled water. Of this 100 c.cs., 50 c.cs. are titrated with standard Soap solution as before, and the amount of Permanent Hardness is calculated in the same manner as described above.

(iii) *Temporary Hardness*—The difference in results between the total hardness and the permanent hardness gives the temporary hardness present in the sample.

When the sample of water is rather soft, Soap solution of 1 c.c. $\equiv$1 mg. of $CaCO_3$ should be used otherwise the second solution is to be used.

Iron.

Apparatus.	*Chemicals.*
Conical Flask.	Dilute Sulphuric Acid (1 : 3).
Dropper.	Potassium Permanganate Solution.

Apparatus.	*Chemicals.*
Stove or Bunsen Burner.	Potassium Ferrocyanide Solution.
Beakers.	Ferric Chloride Solution (1 c.c. $\equiv$ 0·1 mg. Fe.).
Funnels.	Distilled water.
Filter Papers.	
Measuring Cylinders.	
Nessler Glasses.	

Procedure.

Take 100 c.cs. of the sample of water in a conical flask, and add to it few drops of iron free Sulphuric Acid (1 : 3). Then add drop by drop Potassium Permanganate solution till it imparts a permanent pink colour to the mixture. Heat it on an water-bath for some time. If the pink colour disappears on heating, more permanganate should be added and the heating should be repeated to test the permanency of the pink colour. Filter it off and dilute it to 100 c.cs. with distilled water. Allow it to cool. Of this, take 50 c.cs. of the mixture, and add to it 1 c.c. of freshly prepared Potassium Ferrocyanide solution, which colours the mixture *blue* in presence of Iron.

Take 0·5, 0·75 and 1·00 c.c. of standard Ferric Chloride solution (1 c.c. $\equiv$ 0·1 mg. of Iron) in three nessler glasses. Add to each of these glasses a little of iron free Sulphuric Acid and freshly prepared Potassium Ferrocyanide and dilute the volume to 50 c.cs. with iron-free distilled water. Allow it to stand for few minutes to develop the blue colour. Match the blue colour of the mixture obtained before with these standards in the same way, as was done for saline and free ammonia. Suppose, it matches with the standard between 1·00 and 0·75, say with 0·80.

If the blue colour obtained with addition of Potassium Ferrocyanide solution to the sample be too deep, then 50 c.cs. of the sample is diluted to 100 c.cs. of which 50 c.cs. are matched with the standards.

Calculation.

50 c.cs. of the sample	...	$\equiv$ 0·8 c.c. of standard $FeCl_3$ solution,
		$\equiv 0{\cdot}8 \times 0{\cdot}1$ mg. of Fe,
∴ 100 c.cs. ...	...	$\equiv 0{\cdot}8 \times 0{\cdot}1 \times 2$ mg. of Fe,
∴ parts per 100,000	...	$= 0{\cdot}16$ gram of Iron.

Sodium Carbonate.

Apparatus.	*Chemicals.*
Beakers.	Deci-Normal Sulphuric Acid.
Measuring Cylinders.	Phenophthalein Solution.
Burette.	
Pipette.	

Procedure.

When water contains Sodium Carbonate—the sample will be distinctly alkaline, and no permanent hardness will be present in that case.

Take 50 c.cs. of the sample and titrate it with deci-normal Sulphuric Acid using Phenophthalein solution as an indicator.

Then, calculate parts per 100,000 and express this in terms of Sodium Carbonate.

The estimation of Sodium Carbonate can be determined from the filtrate obtained after the Carbonates of Calcium and Magnesium have been thrown down by boiling the sample.

Acidity or Alkalanity.

Apparatus.	*Chemicals.*
Beakers.	Deci-Normal Sodium Carbonate Solution.
Measuring Cylinders.	
Burette.	Deci-Normal Sulphuric Acid.
Pipette.	Methyl Orange Solution.
Funnels.	
Filter Papers.	

Procedure.

Take 25 c.cs. of the filtered sample and titrate it with an alkali of known strength (Deci-normal Sodium Carbonate is used here) for acidity estimation and for alkali, titrate with an acid of known strength using Methyl Orange as an indicator in both cases.

The amount of acid or alkali present in the sample is calculated, the strength of acid or alkali being known, and this is expressed in terms of Sulphuric Acid ; and Sodium Carbonate, or Calcium Carbonate or Ammonia which is likely to be present for acid and alkali respectively or both in terms of Sulphuric Acid and in parts per 100,000.

Free Alkalinity.

Apparatus.	*Chemicals.*
Beaker.	Deci-Normal Sulphuric Acid.
Measuring Cylinder.	Phenophthalein Solution.
Burette.	
Pipette.	

Procedure.

Titrate 50 c.cs. of the sample with deci-normal Sulphuric Acid using Phenolphthalein as an indicator.

Calculate the amount present in terms of Sodium Carbonate in parts per 100,000.

Free Chlorine.

Apparatus.	*Chemicals.*
Chemical Balance with necessary weights.	Distilled Water.
Mortar and Pestle.	Ortho-Tolidine solution.
Nessler Tubes.	Copper Sulphate solution.
Thermometer.	Potassium Dichromate solution.
Stop watch.	
Measuring cylinders.	

Preparation of Standered Solutions.*

Ortho-Tolidine Solution—Dissolve 1 g. of Ortho-tolidine, (melting point 129°C), in a litre of dilute hydrochloric acid (100 ml. of concentrated acid diluted to 1 litre). Pulverizing the Ortho-tolidine will aid in solution.

Solution of the Ortho-tolidine may be facilitated by the following procedure.

1. Weigh out 1 g. of Ortho-tolidine, transfer it to a 6 inch motar, and add 5 ml. of 1 : 5 hydrochloric acid (previously prepared by adding 100 ml. of concentrated hydrochloric acid, sp. gr. 1·18 – 1·19, to 400 ml. of distilled water).

2. Grind to a thin paste and add 150 to 200 ml. of distilled water. The Ortho-tolidine goes into solution immediately.

3. Transfer to a 1000 ml. graduate and make up to 505 ml. with distilled water.

4. Make up to the 1000 ml. mark by adding the balance (495 ml.) of the 1 : 5 hydrochloric acid.

Copper Sulphate Solution—Dissolve 1 : 5 g. of Copper Sulphate and 1 ml. of concentrated sulphuric acid in distilled water and make up to 100 ml.

Potassium Dichromate Solution—Dissolve 0·25 g. of Potassium Dichromate and 1·0 ml. of concentrated sulphuric acid distilled water and make up to a litre.

Ortho-Tolidine Test.

Procedure—Mix 1 ml. of the Ortho-tolidine reagent with 100 ml. of the sample to be tested in a 100 ml. Nessler tube and allow the mixture to stand until the maximum colour has developed. *In no case shall the period allowed for colour development be less than 5 minutes.*

In the case of waters at temperatures below 10°C., those which have been treated with ammonia or ammonium com-

* Extracted from the Standard Methods of Water Analysis, American Public Health Association. (7th Edn. Page, 44-47).

pounds, those containing turbidity or those containing ferric iron, the colour development is retarded and the sample should be allowed to stand for at least 30 minutes after the Ortho-tolidine is added and before the reading is made.

At no time from the taking of the sample to the reading of the colour developed should the sample be exposed to direct sunlight.

Small amounts of free chlorine give a yellow, and larger amounts an orange colour. For quantitative estimation, compare the colour developed with that of standards in tubes prepared from the solutions of copper sulphate and potassium dichromate as indicated in the following table.

TABLE.

Preparation of Permanent Standards for Content of Chlorine.

Chlorine in parts per million.	Solution of Copper Sulphate ml.	Solution of Potassium Dichromate in ml.
0·01	0·0	0·8
0·02	0·0	2·1
0·03	0·0	3·2
0·04	0·0	4·3
0·05	0·4	5·5
0·06	0·8	6·6
0·07	1·2	7·5
0·08	1·5	8·2
0·09	1·7	9·0
0·10	1·8	10·0
0·20	1·9	20·0
0·25	1·9	25·0
0·30	1·9	30·0
0·35	1·9	34·0
0·40	2·0	38·0
0·50	2·0	45·0
0·60	2·0	51·0
0·70	2·0	58·0
0·80	2·0	63·0
0·90	2·0	67·0
1·00	2·0	72·0

When the colour developed does not match the tints of the standards (as may be the case in certain lime treated or natural waters) the addition of more acid is required.

When using the standards listed above, colour comparisons shall be made in 100 ml. Nessler tubes having the graduation mark at 300 mm. from the bottom. If the 100 ml. tubes used have the graduation mark at 240 mm., the colour standards developed by Muer & Hale should be used.

Variations in the height of the graduation mark in any set of colour comparison tubes used in this determination shall not be more than 10 mm.

In preparing the standards, after mixing the quantities of copper sulphate and dichromate solutions as given in the table, dilute with distilled water to the 100 ml. mark.

The use of prepared glass or liquid standards is permissible only when they have been properly checked against the above permanent standard solutions given in the table.

Hydrogen Ion Concentration.*

The *Hydrogen Ion concentration* is an expression of the intensity factor of acid or alkaline properties as opposed to the quantity factors *acidity* and *alkalinity*.

Since by the mass law the product of the hydrogen ion concentration times the hydroxyl ion concentration is equal to a constant, the acid and alkaline intensities may both be expressed in terms of the hydrogen ion concentration.

Numerically this expression is given as either the number of moles of ionized hydrogen per litre, or more conveniently, by means of the negative logarithm of the number of moles per litre, namely, by the so-called pH scale. The table below may be used to convert results in one system to the other.

---

* Extracted from Standard Methods of Water Analysis, American Public Health Association. (7th Edn. Page, 40-41).

TABLE.

Corresponding Values of pH and H-ion concentration for any pH interval.

pH.	Hydrogen Ion concentration.
n·00	$1{\cdot}00 \times 10^{-n}$
n·05	$8{\cdot}91 \times 10^{-(n+1)}$
n·10	7·94
n·15	7·07
n·20	6·31
n·25	5·62
n·30	5·01
n·35	4·46
n·40	3·98
n·45	3·54
n·50	3·16
n·55	2·82
n·60	2·51
n·65	2·24
n·70	1·99
n·75	1·78
n·80	1·58
n·85	1·41
n·90	1·26
n·95	1·12
(n+1)·00	1·00

Example:—Find the H-ion concentration corresponding to pH=6·55,* n=6, n+1=7. From the table H-ion concentration $= 2{\cdot}82 \times 10^{-7}$.

Preparation of Standard Buffer Solutions.*

Deci-Normal Hydrochloric Acid Solution—The solution should be a carefully prepared *exact* deci-normal solution of Hydrochloric Acid.

* Extracted from the Methods of Chemical Analysis as Applied to Sewage and Sewage Effluents. (Ministry of Health Publication, page, 39-40).

Deci-Normal Sodium Hydroxide Solution —This solution must be *Carbonate-free exact* decinormal solution of Sodium Hydroxide.

Glycocoll Solution —This is a tenth molecular Glycocoll solution containing sodium chloride ; 7·505 grams of Glycocoll and 5·85 grams of sodium chloride per litre.

Primary Potassium Phosphate Solution —It is an M/15 solution of primary Potassium Phosphate containing 9·078 grams of KH_2PO_4 per litre.

Secondary Sodium Phosphate Solution—It is an M/15 solution of secondary Sodium Phosphate containing 11·876 grams of Na_2HPO_4, $2H_2O$ per litre.

Secondary Sodium Citrate Solution —It is a tenth molecular solution of secondary Sodium Citrate made from a solution containing 21·008 grams crystallised Citric Acid and 200 c.cs. *carbonate-free* normal Sodium Hydroxide, and diluted to a litre with CO_2-free distilled water.

Alkaline Borate Solution —It is made from 12·404 grams of Boric Acid dissolved in 100 c.cs. carbonate-free normal Sodium Hydroxide and diluted to a litre with CO_2-free distilled water.

Colorimetric Determination.*

Apparatus.

Test Tubes filled with corks.

Day light lamp.

A box fitted with opal glass.

Paraffin Wax.

Chemicals.

Deci-Normal Hydrochloric Acid Solution.

Deci-Normal Sodium Hydroxide Solution (Carbon Free).

Glycocoll Solution.

M/15 Primary Potassium Phosphate Solution.

M/15 Secondary Sodium Phosphate Solution.

Secondary Sodium Citrate Solution.

Alkaline Borate Solution.

Mixtures of various pairs of these solutions are made in a series of test tubes of approximately uniform diameter, the final volume in each tube being 10 c.cs. When the various mixtures have been completed, there results a series of solutions having a range varying by small fractions from pH 1·038 to pH 13·066. To each of these tubes is added the necessary amount of the appropriate indicator; the tubes are then provided with well fitting corks cut off flush with the top of the tube, the top is immersed in hot paraffin wax, and finally a small label is stuck on the tube, giving the pH figure of that particular solution. If smaller tubes are preferred, 5 c.cs. instead of 10 c.cs. may be made up.

Procedure—5 c.cs. or 10 c.cs. (according to the size of buffer tubes used) are placed in a test tube in a rack, the amount of the appropriate indicator added, and the colour matched against those of the standard buffer tubes. In the case of the coloured or turbid liquids care must be taken to place a blank tube of the sample behind the buffer tube. When constant uniformity of light is required, the colour-matching may be done in front of a daylight lamp, the effect being better if the daylight bulb is placed in a box, the front of which consists of a sheet of opal glass.

The solutions used for the determination of hydrogen ion concentration are called *Standard Buffer Solutions* and the tubes *Buffer Tubes.*

The water to be used for the solutions must be boiled, CO_2-free distilled water, and the solutions must be protected against contamination by CO_2.

APPENDIX II.

SAND ANALYSIS BY MEANS OF SIEVES.

(After Allen Hazen).

From 100 to 1200 grams of the sample of sand are taken and dried on an oven at a temperature somewhat above the boiling point. The dry sand is necessary for making it suitable for sifting. The drying can be done in small iron or porcelain dish. After drying, the sample of sand is weighed on a chemical balance.

The sieves required for sand analysis must be made of selected brass wire gauze having as nearly as practicable equal-sized square meshes. The frames are to be of metal, and to be of such diameter, so that the whole set can be loosely rested together. This is necessary to avoid loss of sand when shaking. For the purpose of shaking, the sieves are arranged on mechanical shaker according to coarseness, the top sieve being the coarsest of the series. A definite number of shakes are given, the number is found by experience and depends upon the character of the sand. It is not necessary that the shaking is to be continued until sand ceases passing, but the operation is to be carried on until only a small amount is passing. The sieves are then taken apart, and the portion passing the finest weighed. The process is repeated until all the sand is on the scale, when it should be equal to the original weight. Percentages finer than the size of the meshes of the different sieves are then plotted on a logarthimic paper with the percentages and diameter of sand as abscissae and ordinates, and a curve is obtained from which the required data is found out. A series of sieves with 16, 20, 25, 30, 35, 40, 60, 80, 100, 140 and 200,

meshes to an inch have been found to be satisfactory for sand analysis.

Effective Size—The size of the sand grain is such that 10 per cent. by weight of particles are smaller and 90 per cent. larger than itself.

Uniformity Co-efficient—The ratio of the size of the sand grain, which has 60 per cent. of the sample finer than itself bears to the size which has 10 per cent. finer than itself.

The following form is usually adopted for sand analysis.

Size of Sieve.	Size of Mesh.	Amount passing in grams.	Percent of total weight.
200	0·091	1·05	0·7
160	0·135	3·60	2·4
100	0·182	15·50	10·33
80	0·247	38·75	25·83
60	0·320	80·66	53·74
40	0·460	105·50	70·33
35	0·550	120·10	80·06
30	0·650	140·30	95·53
25	0·930	146·25	97·50
20	1·030	148·80	99·20
16	1·410	149·90	99·93
	Total ...	149·90	99·93
	Loss ...	0·10	00·07
	Amount sieved	150·00	100·00

60% Finer than ... ... 0·38 mm.

10% Finer than effective size ... ... 0·75 mm.

Uniformity co-efficient. ... ... 2·1

The plotting is shewn in Chart IX.

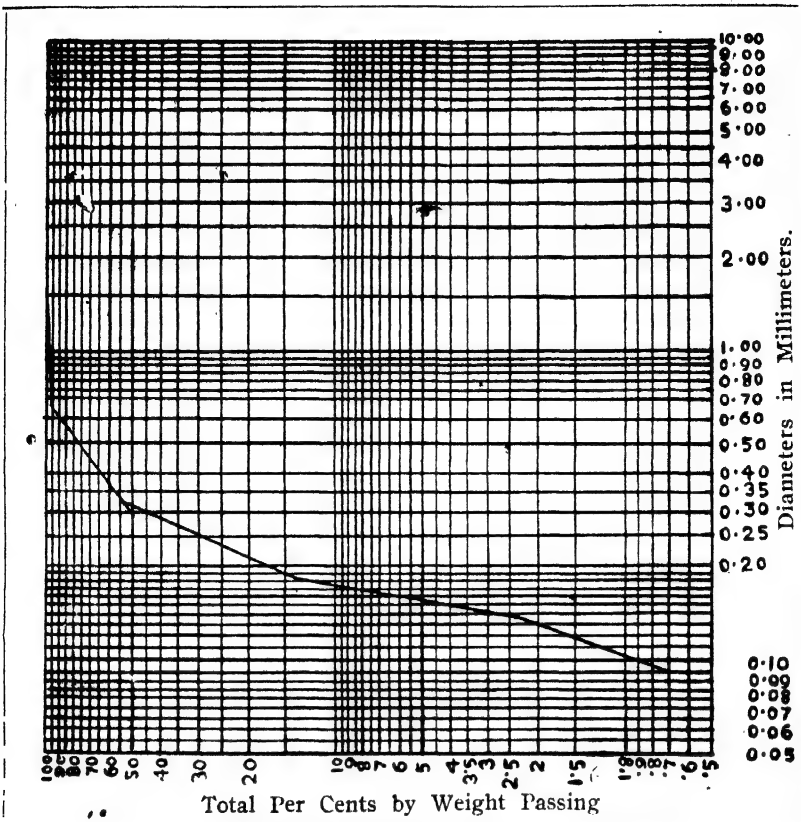


Chart IX—Sand Analysis.

APPENDIX III.

ALKALINITY.

Description of a method of estimating the Alkalinity (total Carbonates) of water with Methyl Orange Tablets and Sodium Acid Sulphate ($NaHSO_4$) Tablets.

Methyl Orange—Methyl orange is used to color the water a light yellow. The amount used does not affect the result so far as the estimation of the alkalinity is concerned, provided the yellow color given to the water is neither too light nor too dark. The methyl orange solution referred to above is prepared by dissolving 1 gram of methyl orange indicator powder in 1000 cubic centimeters of distilled water. The methyl orange tablets are made by mixing a suitable quantity of methyl orange indicator powder with a neutral substance.

The quantity of methyl orange indicator powder for each methyl orange tablet is based upon the addition of 4 drops of methyl orange solution to 100 c.cs. of raw water in a dish glazed white inside about 4¼ inches in diameter and 1⅜ inches deep having a flat bottom, the depth of the water being about ⅜ of an inch. The dish referred to is of the same kind as those sent with the tablets.

The degree of the yellow color to be brought about with methyl orange is, in a sense, a matter of personal preference, as some desire to add more methyl orange to the raw water and obtain a dark yellow, and others wish to add less methyl orange and obtain a light yellow. The depth of water in a dish also bears an important part, as, with the same quantity of methyl orange, the deeper the water, within certain limits, the darker yellow the water appears. As stated, 100 c.cs. of water in the dish referred to had a depth of about ⅜ of an inch, and during laboratory experiments 1 methyl orange tablet colored the water to a satisfactory degree of yellow.

Method of Estimating the Alkalinity—Pour 100 c.cs. of the water to be tested into a dish glazed white inside and add 1 methyl orange tablet which is to be thoroughly dissolved in the water, which will result in a yellow color being given to the water.

The sodium acid sulphate tablets are then placed in the water to be tested, one at a time, and triturated with a pestle until entirely dissolved. The sodium acid sulphate tablets are added until an acid reaction is reached, which is shown by the yellow changing to a pinkish color.

In adding the sodium acid sulphate tablets, be sure that the first one is thoroughly dissolved in the water before adding the second, and so on, accurately noting the number of tablets used to bring about the acid reaction. The final tablet will generally carry the acid reaction beyond the true end point (first indication of pinkish color), so an additional amount of the water to be tested is added, slowly and carefully, to the 100 c.cs. already in the dish, until an exact alkaline reaction is reached, which is when the original yellow color is restored.

The total amount of water in the dish is then measured and 1 c.c. added to the result to allow for water adhering to the inside of the dish, or, to obtain the total amount in another way, the additional water used to produce the exact alkaline reaction can be measured as it is put into the dish and added to the 100 c.cs. In the latter case, 1 c.c. need not be added to allow for water adhering to the inside of the dish.*

$$\frac{1000nA}{W}$$

n = number of sodium acid sulphate tablets used.

W = number of cubic centimeters of water tested to which the methyl orange tablet and sodium acid sulphate tablets have been added.

A = equivalent of each sodium acid sulphate tablet in milligrams of $CaCO_3$.

The equivalent for each sodium acid sulphate tablet is

always stated in the tablet case, which holds the glass tubes in which the tablets are packed.

Blank to be used—To be certain that the original yellow color is restored (which is when the exact alkaline is reached), a comparison should be made with a blank (the same volume of the water in a duplicate dish having one methyl orange tablet dissolved in it). The blank should be prepared at the time that the methyl orange tablet is added to the water to be tested.

Care Required to Restore Original Yellow Color—While it is usually desirable to estimate the alkalinity as accurately as possible, it should be remembered that a positively safe result is the most satisfactory; therefore it is essential that enough water be surely added to change the pinkish color back to the original yellow color, as shown by the yellow color of the blank. The tendency, until one has become accustomed to the delicate colors, is to not add quite enough water, thus leaving the restored yellow color a little darker than the blank, but it is better to add a little too much water than not enough, although there is really no necessity for any error if care is taken, as the greater the quantity of water used in working out the formula, the lower the alkalinity figures, which is on the safe side.

Alkalinity Represented by One or More Tablets—Assuming that sodium acid sulphate tablets have an equivalent of 0·50 milligram of $CaCO_3$, one tablet, in 100 c.cs. of yellow-colored water, will represent an alkalinity of 5·0 parts per million. If 2 sodium acid sulphate tablets have been dissolved in the water and the yellow color of the water has not changed to a pinkish color, it would be known that although the end point had not been reached, the alkalinity was at least 10·0 parts per million; but if, after a third sodium acid sulphate tablet has been dissolved in the water, the yellow color has changed to a pinkish color, it would be known that 15·0 was too high, and to ascertain the exact alkalinity, the original yellow color of the water must be restored, as heretofore

described, by adding more of the water to be tested to the water already in the dish.

Minimum Alkalinity—The number of sodium acid sulphate tablets added and dissolved in the water without producing a change from the yellow to the pinkish color should always be carefully noted and a safe minimum alkalinity worked out for them by the formula, in addition to the exact alkalinity for the total number of sodium acid sulphate tablets used.

Minimum Alkalinity at Time Sufficient—The minimum alkalinity obtained in the manner just mentioned will at times answer all requirements, as in 100 c.cs. of water each sodium acid sulphate tablet (if the equivalent is 0·50) only represents 5·0 parts of alkalinity per million. For example:—If 6 sodium acid sulphate tablets were used, and the yellow color had not changed to a pinkish color after the fifth tablet had been dissolved, it would positively be known that the alkalinity was at least 25, and if the yellow color had changed to a pinkish color when the sixth tablet had been dissolved, it would be known that 30 was too high for the alkalinity and that the exact alkalinity must be between 25 and 30, the 25 being absolutely safe. In estimating the alkalinity in this way, a blank need not be used and the only measurement of water required would be that of the 100 c.cs. first put into the dish.

Variation in Equivalents—The equivalent of the sodium acid sulphate tablets may vary slightly in the manufacture of different lots of tablets; therefore, the glass tubes containing the sodium acid sulphate tablets and the memoranda in the tablet case should always be carefully examined before the tablets are used in order to be sure in regard to the equivalent.

As indicated above, the parts per million of alkalinity represented by each tablet in 100 c.cs. of water is 10 times the equivalent of the tablet.

All parts are per million.

APPENDIX IV.

Dimensions and Weights of Standard Cast Iron and Steel Pipes.

Cast Iron Pipes.

Standard Weights and Dimensions of Straight Spigot & Socket and Flange Pipes.

Nominal Internal Diameter	Thickness	WATER AND SEWAGE											
		Socket and Spigot						Flange					
		Class B.						Class B.					
		Length (exclusive of depth of socket)						Length (measumed over the flanges)					
		Weights						Weights					
		9 feet.			12 feet.			9 feet.			12 feet.		
		Cwts.	qrs.	lbs.	Cwts.	qrs.	lbs.	Cwts.	qrs.	lbs.	Cwts.	qrs.	lbs.
3	·38	1	0	17				1	0	12			
4	·39	1	2	4	1	3	26	1	1	26	1	3	21
5	·41	2	0	4	2	2	14	1	3	21	2	2	4
6	·43	2	2	6	3	1	4	2	1	19	3	0	18
7	·45	3	0	9	3	3	26	2	3	19	3	3	8
8	·47	3	2	24	4	3	4	3	1	27	4	2	7
9	·49	4	1	9	5	2	9	4	0	8	5	1	8
10	·52	5	0	6	6	2	2	4	3	8	6	1	4
12	·57	6	1	20	8	1	6	6	0	14	8	0	1
14	·61				10	1	15				10	0	5
15	·63				11	1	21				11	0	7
16	·65				12	2	7				12	0	18
18	·69				15	0	5				14	2	1
20	·73				17	2	11				17	0	11
21	·75				18	3	25				18	1	21
22	·77				20	2	8				19	3	8
24	·80				23	1	0				22	1	26

Steel Pipes.

Nominal Internal Diameter.	Thickness of Shell.	Packing Space.	Depth of Socket.	Weight Coated and Wrapped.	Weight Coated only.	Weight of Tube Only.
Inches	Inches	Inches.	Inches.	lbs. per foot.	lbs. per foot.	lbs. per foot.
2	0·118	0·295	3·74	3·36	2·76	2·65
3	0·138	0·295	3·74	5·24	4·70	4·59
4	0·157	0·295	4·33	7·86	7·12	6·92
5	0·157	0·295	4·53	9·8	8·80	8·58
6	0·177	0·295	4·53	13·1	11·88	11·63
7	0·197	0·295	4·72	16·8	15·39	15·03
8	0·216	0·315	4·92	21·0	19·29	18·81
9	0·256	0·315	5·12	27·8	25·66	25·20
10	0·276	0·334	5·31	32·9	30·71	29·06
11	0·295	0·334	5·71	39·0	36·16	34·71
12	0·305	0·334	5·91	44·4	40·72	39·78
13	0·315	0·334	6·10	49·1	45·7	45·12
14	0·315	0·334	6·30	53·8	49·7	47·80
15	0·354	0·354	6·30	65·9	61·5	57·61
16	0·394	0·374	6·30	73·9	69·6	68·40
17	0·413	0·394	6·50	83·0	78·9	76·23
18	0·453	0·413	6·50	96·4	91·4	88·61
19	0·472	0·433	6·50	104·7	100·3	97·42
20	0·492	0·453	6·70	114·6	109·9	106·63

APPENDIX IV.

DIMENSIONS AND WEIGHTS OF STANDARD CAST IRON AND STEEL PIPES.

Standard 90° Bends.

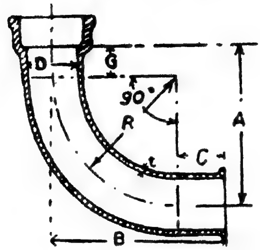
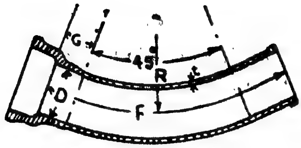
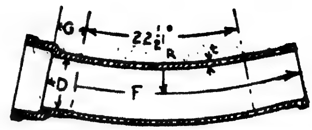


TABLE I.

D Nominal Internal Diameter.	A	B	C	G	R	Classes A and B. Thickness t	Classes A and B. Weight. Socket & Spigot ends.
ins.	fts ins.	ft. ins.	ft. ins.	ins.	ft. ins.	in.	cwt. Qrs. lbs.
3	1 0	1 3	0 6	3	0 9	·38	0 1 13
4	1 3	1 6	0 6	3	1 0	·39	0 2 4
5	1 3½	1 6	0 6	3½	1 0	·41	0 2 27
6	1 6½	1 9	0 6	3½	1 3	·43	1 0 4
7	1 6½	1 10	0 7	3½	1 3	·45	1 1 1
8	1 10	2 2	0 8	4	1 6	·47	1 3 0
9	1 10	2 3	0 9	4	1 6	·49	2 0 7
10	2 1	2 7	0 10	4	1 9	·52	2 2 22
12	2 1	2 9	1 0	4	1 9	·57	3 2 1
14	2 1½	2 9	1 0	4½	1 9	·61	4 1 23
15	2 4½	3 0	1 0	4½	2 0	·63	5 0 27
16	2 4½	3 0	1 0	4½	2 0	·65	5 3 0
18	2 4½	3 0	1 0	4½	2 0	·69	6 3 20
20	2 7½	3 3	1 0	4½	2 3	·73	8 2 13
21	2 10½	3 6	1 0	4½	2 6	·75	9 3 15
22	3 2	3 9	1 0	5	2 9	·77	11 2 21
24	3 5	4 0	1 0	5	3 0	·80	13 2 23

Standard 45°, 22½° and 11¼° Bends.





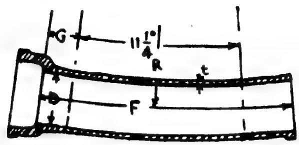


TABLE II.

D Nominal Internal Diameter.	F	G	R 45°	R 22½°	R 11¼°	Classes A and B. Thickness t	Classes A and B. Weight. Socket & Spigot ends.
ins.	ft. ins.	ins.	ft. ins.	ft. ins.	ft. ins.	in.	cwt. Qrs. lbs
3	2 4	3	2 0	4 0	8 0	·38	0 1 18
4	2 4	3	2 0	4 0	8 0	·39	0 2 5
5	2 7	3½	2 3	4 6	9 0	·41	0 3 4
6	2 7	3½	2 3	4 6	9 0	·43	0 3 27
7	2 9	3½	2 6	5 0	10 0	·45	1 0 27
8	2 10	4	2 6	5 0	10 0	·47	1 2 6
9	3 0	4	2 9	5 6	11 0	·49	1 3 14
10	3 0	4	2 9	5 6	11 0	·52	2 0 25
12	3 3	4	3 0	6 0	12 0	·57	2 3 23
14	3 6	4½	3 3	6 6	13 0	·61	3 3 22
15	3 8	4½	3 6	7 0	14 0	·63	4 1 25
16	3 8	4½	3 6	7 0	14 0	·65	4 3 17
18	3 10	4½	3 9	7 6	15 0	·69	6 0 13
20	4 1	4½	4 0	8 0	16 0	·73	7 1 25
21	4 3	4½	4 3	8 6	17 0	·75	8 1 6
22	4 4	5	4 3	8 6	17 0	·77	9 1 0
24	4 6	5	4 6	9 0	18 0	·80	10 2 27

Standard Tees.

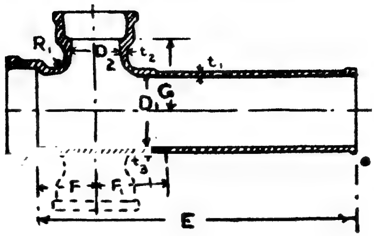


TABLE III.

The Radius R, shall be 1½ inches for all sizes of Tees.

Nominal Internal Diameter.		E	F and F_1	G	Classes A and B.			
					Thickness.			Weight.
D_1	D_2				t_1	t_2	t_3	Socket and Spigot ends.
ins.	ins.	ft. ins.	ft ins.	ft. ins.	in.	in.	in.	cwt. qrs. lbs.
3	3	3 0	0 5	0 5	·38	·57	·57	0 2 27
4	3	3 0	0 5	0 6	·39	·57	·58	0 3 19
	4	3 0	0 6	0 6	·39	·58	·58	0 3 27
5	3	3 0	0 5	0 6	·41	·57	·60	1 0 17
	4	3 0	0 6	0 6	·41	·58	·60	1 0 25
	5	3 0	0 7	0 7	·41	·60	·60	1 1 14
6	3	3 0	0 6	0 7	·43	·57	·62	1 1 18
	4	3 0	0 6	0 7	·43	·58	·62	1 1 25
	5	3 0	0 7	0 7	·43	60	·62	1 2 11
	6	3 0	0 7	0 7	·43	·62	·62	1 2 21
7	3	3 0	0 6	0 7	·45	·57	·64	1 2 16
	4	3 0	0 6	0 8	·45	·58	·64	1 2 24
	5	3 0	0 7	0 8	·45	60	·64	1 3 11
	6	3 0	0 7	0 8	·45	·62	·64	1 3 22
	7	3 0	0 8	0 8	·45	·64	64	2 0 6
8	3	3 6	0 6	0 8	·47	·57	·66	2 0 17
	4	3 6	0 6	0 8	·47	·58	·66	2 0 23
	5	3 6	0 7	0 8	·47	·60	·66	2 1 10
	6	3 6	0 7	0 8	·47	·62	·66	2 1 20
	7	3 6	0 8	0 8	·47	·64	·66	2 2 4
	8	3 6	0 8	0 8	·47	66	·66	2 2 21

Standard Tees—(Contd.)

Nominal Internal Diameter. D_1	D_2	E		F and F_1		G		Classes A and B. Thickness. t_1	t_2	t_3	Weight. Socket and Spigot ends.		
ins	ins.	ft.	ins.	ft.	ins.	ft.	ins.	in.	in.	in.	cwt.	qrs.	lbs.
9	3	3	6	0	6	0	8	·49	·57	·68	2	1	23
	4	3	6	0	6	0	9	49	·58	·68	2	2	3
	5	3	6	0	7	0	9	·49	·60	·68	2	2	18
	6	3	6	0	7	0	9	·49	·62	·68	2	3	1
	7	3	6	0	8	0	9	·49	·64	·68	2	3	13
	8	3	6	0	8	0	9	·49	·66	·68	3	0	2
	9	3	6	0	9	0	9	·49	·68	·68	3	0	17
10	3	3	6	0	6	0	9	·52	·57	·71	2	3	14
	4	3	6	0	6	0	9	·52	·58	71	2	3	20
	5	3	6	0	7	0	9	·52	·60	·71	3	0	9
	6	3	6	0	7	0	9	·52	·62	·71	3	0	19
	7	3	6	0	8	0	9	·52	·64	·71	3	1	6
	8	3	6	0	9	0	9	·52	66	·71	3	1	27
	9	3	6	0	9	0	10	·52	·68	·71	3	2	12
	10	3	6	0	10	0	10	·52	·71	·71	3	3	2
12	3	3	6	0	6	0	10	·57	·57	·76	3	2	7
	4	3	6	0	7	0	10	·57	·58	·76	3	2	17
	5	3	6	0	7	0	10	·57	·60	76	3	3	3
	6	3	6	0	8	0	10	·57	·62	·76	3	3	18
	7	3	6	0	8	0	11	·57	64	·76	4	0	1
	8	3	6	0	9	0	11	·57	·66	·76	4	0	23
	9	3	6	0	9	0	11	·57	·68	·76	4	1	7
	10	3	6	0	10	0	11	·57	·71	76	4	1	27
	12	3	6	0	11	0	11	·57	76	·76	4	2	27
14	4	4	0	0	7	0	11	·61	58	·80	4	3	20
	6	4	0	0	8	1	0	·61	·62	·80	5	0	21
	8	4	0	0	9	1	0	·61	·66	·80	5	1	27
	10	4	0	0	10	1	0	·61	·71	·80	5	3	3
	12	4	0	0	11	1	0	·61	·76	·80	6	0	4
	14	4	0	1	0	1	0	·61	·80	·80	6	1	19
15	4	4	0	0	7	1	0	·63	58	82	5	1	13
	6	4	0	0	8	1	0	·63	62	·82	5	2	14
	8	4	0	0	9	1	0	·63	·66	·82	5	3	21
	10	4	0	0	10	1	0	·63	·71	·82	6	0	25
	12	4	0	0	11	1	1	·63	76	·82	6	1	26
	15	4	0	1	1	1	1	·63	·82	·82	7	0	10
16	4	4	0	0	7	1	0	·65	·58	·84	5	3	11
	6	4	0	0	8	1	1	·65	·62	·84	6	0	13

Standard Tees—(Contd.)

Nominal Internal Diameter D_1	Nominal Internal Diameter D_2	E	F and F_1	G	Classes A and B. Thickness. t_1	Thickness. t_2	Thickness. t_3	Weight. Socket and Spigot ends.
ins.	ins.	ft. ins.	ft. ins.	ft. ins.	in.	in.	in.	cwt. qrs. lbs.
16	8	4 0	0 9	1 1	·65	·66	·84	6 1 20
	10	4 0	0 10	1 1	·65	·71	·84	6 2 25
	12	4 0	0 11	1 1	·65	·76	·84	6 3 26
	15	4 0	1 1	1 1	·65	·82	·84	7 2 11
	16	4 0	1 1	1 1	65	·84	·84	7 3 0
18	4	4 6	0 7	1 1	·69	·58	·88	7 2 8
	6	4 6	0 8	1 2	·69	·62	·88	7 3 10
	8	4 6	0 9	1 2	·69	·66	·88	8 0 17
	10	4 6	0 10	1 2	69	·71	·88	8 1 23
	12	4 6	0 11	1 2	·69	·76	·88	8 2 25
	15	4 6	1 1	1 2	·69	·82	·88	9 1 11
	16	4 6	1 2	1 2	69	·84	·88	9 2 7
	18	4 6	1 3	1 3	·69	·88	·88	10 0 19
20	6	4 6	0 8	1 3	73	·62	·92	9 0 3
	8	4 6	0 9	1 3	·73	·66	·92	9 1 11
	10	4 6	0 10	1 3	·73	·71	·92	9 2 17
	12	4 6	1 0	1 3	·73	·76	·92	9 3 27
	15	4 6	1 1	1 3	·73	·82	·92	10 2 7
	16	4 6	1 2	1 3	·73	·84	·92	10 3 4
	18	4 6	1 3	1 4	·73	·88	·92	11 1 16
	20	4 6	1 4	1 4	·73	·92	·92	11 3 6
21	6	4 6	0 8	1 3	·75	·62	·94	9 2 20
	8	4 6	0 9	1 3	·75	·66	·94	10 0 1
	10	4 6	0 10	1 4	·75	·71	·94	10 1 7
	12	4 6	1 0	1 4	·75	·76	·94	10 2 18
	15	4 6	1 1	1 4	·75	·82	·94	11 0 26
	16	4 6	1 2	1 4	·75	·84	·94	11 1 23
	18	4 6	1 3	1 4	·75	·88	·94	12 0 8
	20	4 6	1 4	1 4	·75	·92	·94	12 1 12
	21	4 6	1 4	1 4	·75	·94	·94	12 2 22
22	6	5 0	0 8	1 4	·77	·62	·96	11 1 11
	8	5 0	0 9	1 4	·77	·66	·96	11 2 20
	10	5 0	0 10	1 4	·77	·71	·96	11 3 27
	12	5 0	1 0	1 4	·77	·76	·96	12 1 10
	15	5 0	1 1	1 4	·77	·82	·96	12 3 19
	16	5 0	1 2	1 4	·77	·84	·96	13 0 16
	18	5 0	1 3	1 5	·77	·88	·96	13 3 1
	20	5 0	1 4	1 5	·77	·92	·96	14 0 24
	22	5 0	1 5	1 5	·77	·96	·96	14 3 12

Standard Tees—(Contd.)

Nominal Internal Diameter.		E	F and F_1	G	Classes A and B.			
					Thickness.			Weight.
D_1	D_2				t_1	t_2	t_3	Socket and Spigot ends.
ins.	ins.	ft. ins.	ft. ins.	ft. ins.	in.	in.	in.	cwt. qrs. lbs
24	6	5 0	0 8	1 5	·80	·62	·99	12 2 25
	8	5 0	0 9	1 5	·80	·66	99	13 0 7
	10	5 0	0 11	1 5	·80	·71	·99	13 1 23
	12	5 0	1 0	1 5	·80	·76	·99	13 2 27
	15	5 0	1 1	1 6	·80	·82	99	14 1 8
	16	5 0	1 2	1 6	·80	·84	·99	14 2 6
	18	5 0	1 3	1 6	·80	88	·99	15 0 20
	20	5 0	1 4	1 6	·80	·92	·99	
	24	5 0	1 6	1 6	·80	·99	·99	16 3 2

Standard Tapers.

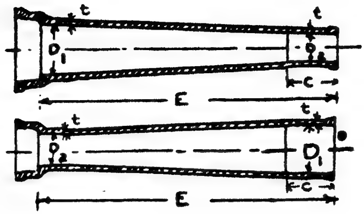
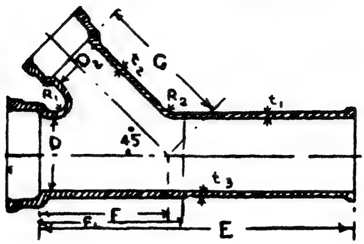


TABLE IV.

Nominal Internal Diameter.		E	C	Classes A and B.		
				Thickness t	Weight.	
D_1	D_2				Type I.	Type 2.
ins.	ins.	ft. ins.	ins.	in.	cwt. qrs. lbs.	cwt. qrs. lbs.
4	3	3 0	6	·39	0 2 8	0 2 6
5	4	3 0	6	·41	0 3 5	0 2 25
5	3	3 0	6	·41	0 2 25	0 2 16
6	5	3 0	6	·43	1 0 0	0 3 24
6	4	3 0	6	·43	0 3 20	0 3 8
6	3	3 0	6	·43	0 3 12	0 2 26
7	6	3 0	6	·45	1 0 24	1 0 21
7	4	3 0	6	·45	1 0 5	0 3 18
8	7	3 0	6	·47	1 2 1	1 1 19
8	6	3 0	6	·47	1 1 19	1 1 5
9	8	3 0	6	·49	1 3 3	1 2 25
9	6	3 0	6	·49	1 2 8	1 1 17
10	9	3 0	6	·52	2 0 9	2 0 4
10	8	3 0	6	·52	1 3 24	1 3 13
12	9	4 0	6	·57	2 2 18	2 2 7
14	12	4 0	6	·61	3 2 14	3 1 22
15	14	4 0	6	·63	4 0 17	4 0 13
15	12	4 0	6	·63	3 3 8	3 2 13
16	15	4 0	6	·65	4 2 7	4 2 0
18	15	4 0	6	·69	5 0 18	4 3 14
20	18	4 0	6	·73	6 1 0	6 0 15
21	20	4 0	6	·75	6 3 24	6 3 14
22	21	4 0	6	·77	7 2 20	7 1 17
24	22	4 0	6	·80	8 1 27	8 1 9

Standard 45° Branches.



The Radii R_1 and R_2 shall $1\frac{1}{2}$ inches for all sizes of branches.

TABLE V.

Nominal Internal Diameter		E	F	F_1	G	Classes A and B. Thickness.			Weight.
D_1	D_2					t_1	t_2	t_3	Socket and Spigot ends.
ins.	ins.	ft. ins.	ft. ins.	ft. ins.	ft. ins.	in.	in.	in.	cwt. qrs. lbs.
3	3	3 0	0 10	1 1	0 10	·38	·57	·57	0 3 10
4	3	3 0	0 11	1 1	0 11	·39	·57	·58	1 0 1
	4	3 0	1 0	1 2	1 0	·39	·58	·58	1 0 13
5	3	3 0	1 0	1 1	1 0	·41	·57	·60	1 1 1
	4	3 0	1 0	1 2	1 0	·41	·58	·60	1 1 13
	5	3 0	1 1	1 4	1 1	·41	·60	·60	1 2 4
6	3	3 6	1 0	1 1	1 1	·43	·57	·62	1 2 0
	4	3 6	1 1	1 3	1 1	·43	·58	·62	1 2 11
	5	3 6	1 2	1 4	1 2	·43	·60	·62	1 3 3
	6	3 6	1 3	1 6	1 3	·43	·62	·62	1 3 21
7	3	3 6	1 1	1 1	1 1	·45	·57	·64	1 2 26
	4	3 6	1 2	1 3	1 2	·45	·58	·64	1 3 11
	5	3 6	1 2	1 4	1 3	·45	·60	·64	2 0 2
	6	3 6	1 3	1 6	1 3	·45	·62	·64	2 0 21
	7	3 6	1 4	1 7	1 4	·45	·64	·64	2 1 13
8	3	3 6	1 1	1 1	1 2	·47	·57	·66	2 0 6
	4	3 6	1 2	1 3	1 3	·47	·58	·66	2 0 18
	5	3 6	1 3	1 4	1 3	·47	·60	·66	2 1 10
	6	3 6	1 4	1 6	1 4	·47	62	·66	2 2 1
	7	3 6	1 5	1 7	1 5	·47	·64	·66	2 2 21
	8	3 6	1 5	1 9	1 5	·47	·66	·66	2 3 20

Standard 45° Branches—(Contd.)

Nominal Internal Diameter.		E	F	F_1	G	Classes A and B.			
						Thickness.			Weight.
D_1	D_2					t_1	t_2	t_3	Socket and Spigot end.
ins.	ins.	ft. ins.	ft. ins.	ft. ins.	ft. ins.	in.	in.	in.	cwt. qrs. lbs.
9	3	3 6	1 2	1 1	1 3	·49	·57	·68	2 1 9
	4	3 6	1 3	1 3	1 4	·49	·58	·68	2 1 22
	5	3 6	1 4	1 4	1 4	·49	·60	·68	2 2 14
	6	3 6	1 4	1 6	1 5	·49	62	·68	2 3 5
	7	3 6	1 5	1 7	1 5	·49	·64	·68	2 3 26
	8	3 6	1 6	1 9	1 6	·49	·66	·68	3 0 25
	9	3 6	1 7	1 10	1 7	·49	68	·68	3 1 21
10	3	4 0	1 3	1 1	1 4	·52	·57	·71	2 2 23
	4	4 0	1 3	1 3	1 4	·52	·58	·71	2 3 8
	5	4 0	1 4	1 4	1 5	·52	·60	·71	3 0 0
	6	4 0	1 5	1 6	1 6	·52	·62	·71	3 0 20
	7	4 0	1 6	1 7	1 6	·52	·64	·71	3 1 13
	8	4 0	1 6	1 9	1 7	·52	·66	·71	3 2 12
	9	4 0	1 7	1 11	1 7	·52	·68	·71	3 3 2
	10	4 0	1 8	2 0	1 8	·52	·71	·71	4 3 6
12	3	4 0	1 4	1 2	1 5	·57	·57	·76	3 1 8
	4	4 0	1 5	1 3	1 6	·57	·58	·76	3 1 22
	5	4 0	1 6	1 5	1 7	·57	·60	·76	3 2 15
	6	4 0	1 6	1 6	1 7	·57	·62	·76	3 3 7
	7	4 0	1 7	1 8	1 8	·57	·64	·76	4 0 1
	8	4 0	1 8	1 9	1 8	·57	·66	·76	4 1 1
	9	4 0	1 9	1 11	1 9	·57	·68	·76	4 1 26
	10	4 0	1 9	2 0	1 10	·57	·71	·76	5 3 3
	12	4 0	1 11	2 4	1 11	·57	·76	·76	6 1 4

Standard Collars.

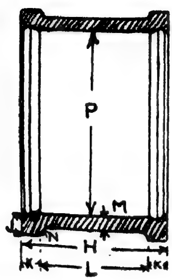


TABLE VI.

Nominal Internal Dimater of pipe.	H	J	K	L	N	Classes A and B.				
						M	P	Weight.		
ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	cwt.	qrs.	lbs.
3	9	1	1	7	7/16	23/32	4⅝	0	1	4
4	9	1	1	7	7/16	25/32	5 11/16	0	1	14
5	10½	1⅛	1⅛	8¼	½	27/32	6 13/16	0	2	7
6	10½	1¼	1¼	8	9/16	29/32	7⅞	0	2	23
7	10½	1¼	1¼	8	9/16	15/16	8 15/16	0	3	7
8	12	1 5/16	1¼	9½	9/16	1 1/32	10	1	0	11
9	12	1 13/32	1⅜	9¼	9/16	1 1/32	11 1/16	1	1	0
10	12	1 15/32	1 7/16	9⅛	9/16	1 1/32	12⅛	1	1	14
12	12	1⅝	1½	9	9/16	1 1/32	14	1	3	27
14	13½	1¾	1⅝	10¼	⅝	1 1/16	16⅛	2	2	16
15	13½	1 25/32	1⅝	10¼	⅝	1 1/16	17⅛	2	3	6
16	13½	1 27/32	1⅝	10¼	⅝	1 3/32	18¼	3	0	8
18	13½	1 31/32	1¾	10	⅝	1 3/16	20¼	3	3	3
20	13½	2 1/16	1⅞	9¾	11/16	1 3/16	22⅜	4	1	6
21	13½	2⅛	1⅞	9¾	11/16	1 7/32	23⅜	4	2	13
22	15	2 3/16	2	11	¾	1 9/32	24½	5	2	3
24	15	2 9/32	2	11	¾	1 5/16	26½	6	0	16

STEEL PIPES DIMENSIONS.
Standard Spigot and Faucet Steel Bends.

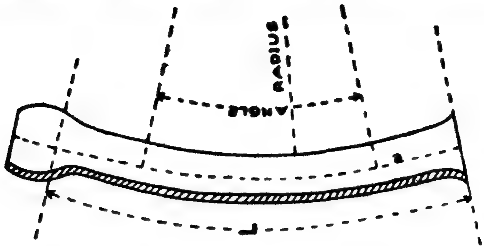


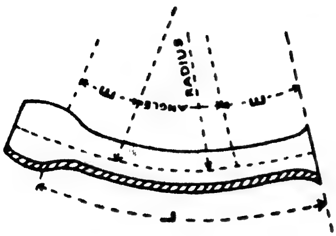
TABLE VII.

Bore.	Radius.		Angle.	Length "L"			
				Ordinary joint.		Rigid or welding joint.	
in.	ft.	in.		ft	in	ft.	in.
1½	1	3	45°	1	8 5/16	1	9½
2	1	8	45°	2	0¼	2	3
2½	2	1	45°	2	4⅛	2	8 7/16
			30°	1	9⅝	2	1 15/16
3	2	6	45°	2	8 1/16	3	1 9/16
			30°	2	0¼	2	5¾
3½	2	11	45°	3	1¼	3	7⅛
			30°	2	4	2	9⅞
			22½°	1	11½	2	5⅜
4	3	4	45°	3	6 7/16	4	1 3/16
			30°	2	8	3	2¾
			22½°	2	2 11/16	2	9½
5	4	2	45°	4	4 13/16	5	0 1/16
			30°	3	3¾	3	11
			22½°	2	9 3/16	3	4 7/16
6	5	0	45°	5	3⅛	5	11 5/16
			30°	3	11⅜	4	7 9/16
			22½°	3	3 9/16	3	11¾
			11¼°	2	3¾	3	0¼
7	5	10	45°	6	1½	6	10
			30°	4	7 3/16	5	3 11/16
			22½°	3	10	4	6½
			11¼°	2	8¼	3	4¾

Standard Spigot and Faucet Steel Bends—(Contd.)

Bore.	Radius.		Angle.	Length "L"			
				Ordinary joint.		Rigid or welding joint.	
in.	ft.	in.		ft.	in.	ft.	in.
8	6	8	45°	6	11⅞	7	8¾
			30°	5	2 15/16	5	11 13/16
			22½°	4	4 7/16	5	1 5/16
			11¼°	3	0¾	3	9⅝
9	7	6	45°	8	1⅜	8	9 3/16
			30°	6	1 13/16	6	9⅝
			22½°	5	2 1/16	5	9⅞
			11¼°	3	8⅜	4	4 1/16
10	8	4	45°	9	0 1/16	9	8 7/16
			30°	6	9 15/16	7	7⅛
			22½°	5	7⅞	6	5 1/16
			11¼°	4	0 3/16	4	9⅜
11	9	2	45°	9	10 13/16	10	7
			30°	7	6	8	2 15/16
			22½°	6	3⅝	6	11 5/16
			11¼°	4	6	5	2 5/16
12	10	0	45°	10	9½	11	6⅜
			30°	8	1⅛	8	10 15/16
			22½°	6	9 7/16	7	7¼
			11¼°	4	9⅞	5	7 11/16

Standard Spigot & Faucet Steel Bend.



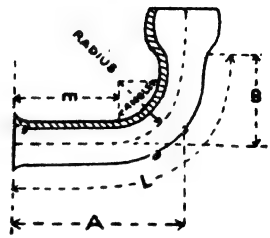


TABLE VIII.

Bore.	Radius.		Angle.		Length "L". Ordinary joint.		Length "L". Rigid or welding joint.	
in.	ft.	in.			ft.	in.	ft.	in.
1½	0	6	90°	(¼)	1	9 15/16	1	10 7/16
			60°		1	5 1/16	1	5 9/16
			45°	(⅛)	1	3 ½	1	4
			30°		1	1 15/16	1	2 7/16
			22·5°	(1/16)	1	1	1	2
			11·25°	(1/32)	0	11 15/16	1	0 7/16
2	0	7½	90°	(¼)	2	0 ¼	2	1 ¼
			60°		1	6 ⅝	1	7 ⅝
			45°	(⅛)	1	4 ⅝	1	5 ⅝
			30°		1	2 ¾	1	3 ¾
			22·5°	(1/16)	1	1 11/16	1	2 11/16
			11·25°	(1/32)	1	0 ¼	1	1 ¼
2½	0	8	90°	(¼)	2	1 1/16	2	2 9/16
			60°		1	7 ⅛	1	9 3/16
			45°	(⅛)	1	5 1/16	1	7 ⅛
			30°		1	2 15/16	1	5
			22·5°	(1/16)	1	1 ⅞	1	3 15/16
			11·25°	(1/32)	1	0 5/16	1	2 5/16
3	0	10	90°	(¼)	2	4 3/16	2	5 15/16
			60°		1	9 ¼	2	0 ½
			45°	(⅛)	1	5 ⅝	1	9 ⅞
			30°		1	4	1	7 ¼
			22·5°	(1/16)	1	2 ¾	1	6
			11·25°	(1/32)	1	0 ¾	1	4

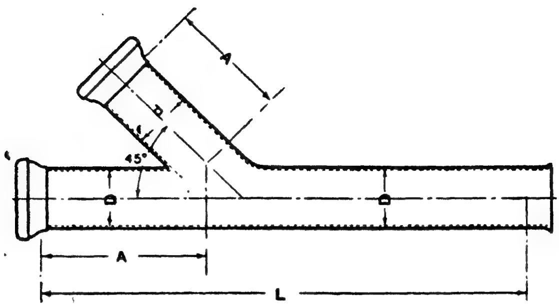
Standard Spigot & Faucet Steel Bend.—Contd.

Bore.	Radius.	Augle.		Length "L".			
				Ordinary joint.		Rigid or welding joint.	
in.	ft. in			ft.	in.	ft.	in.
3½	0 11	90°	(¼)	2	7	2	9
		60°		1	10 ⅞	2	3 ⅛
		45°	(⅛)	1	8	2	0 ½
		30°		1	5 ⅛	1	9 ⅝
		22·5°	(1/16)	1	3 11/16	1	8 3/16
		11·25°	(1/32)	1	1 9/16	1	6 1/16
4	1 0	90°	(¼)	2	9 ⅞	3	0 ⅜
		60°		2	0 9/16	2	6 5/16
		45°	(⅛)	1	9 7/16	2	3 3/16
		30°		1	6 5/16	2	0 1/16
		22·5°	(1/16)	1	4 11/16	1	10 7/16
		11·25°	(1/32)	1	2 ⅜	1	8 ⅛
5	1 5	90°	(¼)	3	8 ¼	3	10 ¾
		60°		2	7 5/16	3	2 9/16
		45°	(⅛)	2	2 ⅞	2	10 ⅛
		30°		1	10 ⅜	2	5 ⅝
		22·5°	(1/16)	1	8 3/16	2	3 7/16
		11·25°	(1/32)	1	4 ⅞	2	0 ⅛
6	1 10	90°	(¼)	4	6 9/16	4	9 13/16
		60°		3	3 1/16	3	11 ¼
		45°	(⅛)	2	9 ¼	3	5 7/16
		30°		2	3 ½	2	11 11/16
		22·5°	(1/16)	2	0 ⅝	2	8 13/16
		11·25°	(1/32)	1	8 ⅜	2	4 9/16
7	2 2	90°	(¼)	5	0 ⅞	5	4 ⅜
		60°		3	11 ¼	4	6 ¼
		45°	(⅛)	2	2 15/16	3	11 7/16
		30°		2	8 ⅛	3	4 ⅝
		22·5°	(1/16)	2	4 ¾	3	1 ¼
		11·25°	(1/32)	1	11 ⅝	2	8 ½
8	2 6	90°	(¼)	5	10 ⅛	6	1 ⅜
		60°		4	4 7/16	5	1 5/16
		45°	(⅛)	3	8 9/16	4	5 7/16
		30°		3	0 11/16	3	9 9/16
		22·5°	(1/16)	2	8 13/16	3	5 11/16
		11·25°	(1/32)	2	2 15/16	2	11 13/16

Standard Spigot & Faucet Steel Bend.—Contd.

Bore.	Radius.	Angle.		Length "L". Ordinary joint.	Length "L". Rigid or welding joint.
in.	ft. in.			ft. in.	ft. in.
9	3 0	90°	(¼)	7 1 5/16	7 5 5/16
		60°		5 4 7/16	6 0 3/16
		45°	(⅛)	4 7	5 2 ¾
		30°		3 9 ⅝	4 5 ⅜
		22·5°	(1/16)	3 4 ⅞	4 0 ⅝
		11·25°	(1/32)	2 9 13/16	3 5 9/16
10	3 6	90°	(¼)	8 1 ½	8 5 ¾
		60°		6 1 ½	6 9 ¾
		45°	(⅛)	5 2 ½	5 10 ¾
		30°		4 4 ½	5 0 ¾
		22·5°	(1/16)	3 10 3/16	4 6 5/16
		11·25°	(1/32)	3 1 ¾	3 10
11	4 6	90°	(¼)	9 11 15/16	10 3 13/16
		60°		7 5	8 1 ¼
		45°	(⅛)	6 2 15/16	6 11 3/16
		30°		5 0 ¾	5 9
		22·5°	(1/16)	4 5 11/16	4 11 5/16
		11·25°	(1/32)	3 7 ⅛	4 3 ⅜
12	5 6	90°	(¼)	11 8 15/16	12 1 15/16
		60°		8 8 ¾	9 5 ¼
		45°	(⅛)	7 3 ⅛	8 0
		30°		5 9 13/16	6 6 11/16
		22·5°	(1/16)	5 1 3/16	5 10 1/16
		11·25°	(1/32)	4 0 3/16	4 9 1/16

Standard Spigot and Faucet Steel Y-pieces or Branches.



The Distance 'A' is equal to D+3″ for all sizes.

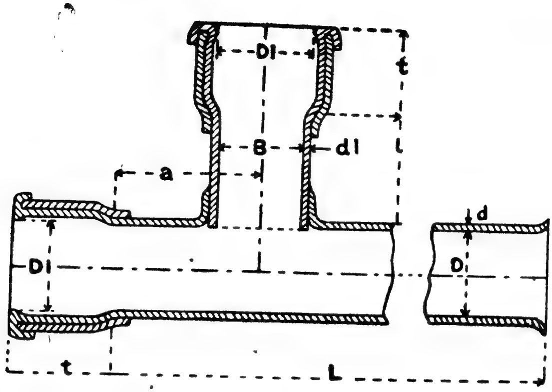
TABLE IX.

Size of Angle Branch piece.		Body D.	Branch d.	Length A.		Overall Length. L.	
in.	in.	in.	in.	ft.	in.	ft.	in.
1½	× 1½	1½	1½	0	4½	2	7½
2	× 1½	2	1½	0	5	2	7½
2	× 2	2	2	0	5	2	7½
3	× 2	3	2	0	6	2	7½
3	× 3	3	3	0	6	2	7½
4	× 2	4	2	0	7	2	7½
4	× 3	4	3	0	7	2	7½
4	× 4	4	4	0	7	3	3
5	× 2	5	2	0	8	3	3
5	× 3	5	3	0	8	3	3
5	× 4	5	4	0	8	3	3
5	× 5	5	5	0	8	3	3
6	× 2	6	2	0	9	3	3
6	× 3	6	3	0	9	3	3
6	× 4	6	4	0	9	3	3
6	× 6	6	6	0	9	3	3
7	× 3	7	3	0	10	3	3
7	× 4	7	4	0	10	3	3

Standard Spigot and Faucet Steel Y-pieces or Branches —(Contd.).

Size of Angle Branch piece.		Body	Branch	Length		Overall Length.	
		D.	d.	A.		L.	
in.	in.	in.	in.	ft.	in.	ft.	in.
7	× 6	7	6	0	10	3	3
7	× 7	7	7	0	10	3	3
8	× 2	8	2	0	11	3	3
8	× 3	8	3	0	11	3	3
8	× 4	8	4	0	11	3	3
8	× 6	8	6	0	11	3	3
8	× 8	8	8	0	11	3	3
9	× 3	9	3	1	0	3	3
9	× 4	9	4	1	0	3	3
9	× 6	9	6	1	0	3	3
9	× 8	9	8	1	0	3	3
10	× 4	10	4	1	1	3	3
10	× 6	10	6	1	1	3	3
10	× 8	10	8	1	1	3	3
11	× 4	11	4	1	2	3	6
11	× 6	11	6	1	2	3	6
11	× 8	11	8	1	2	3	6
12	× 4	12	4	1	3	3	6
12	× 6	12	6	1	3	3	6
12	× 8	12	8	1	3	3	6

Standard Spigot and Faucet Steel T Pieces.



Tees made to full lines when outlet is 1 inch or more smaller than the diameter of main.

When outlet is larger than this, body of Tee will be to dotted lines.

TABLE X.

Size of Tee piece.	For body of Tee piece.		For branch of Tee piece.		Distance from back of Faucet to centre of branch.
	Bore of Tube.	Length	Bore of Branch.	Length from outside of body.	
	D	L	B	t	a
in. in. in.	in	ft, in.	in,	in.	in.
1½ × 1½ × 1½	1½	2 7½	1½	5	5
2 × 2 × 1½	2	2 7½	1½	5	5¼
2 × 2 × 2	2	2 7½	2	5	5½
3 × 3 × 2	3	2 7½	2	5	5½
3 × 3 × 3	3	2 7½	3	5	6¼
4 × 4 × 2	4	2 7½	2	5	5¾
4 × 4 × 3	4	2 7½	3	5	6¾
4 × 4 × 4	4	2 7½	4	5¼	6¾
5 × 5 × 2	5	3 3	2	5	6

Standard Spigot and Faucet Steel T Pieces—(Contd.)

Size of Tee piece.	For body of Tee piece.		For branch of Tee piece.		Distance from back of Faucet to centre of branch.
	Bore of Tube.	Length.	Bore of Branch.	Length from outside of body.	
	D	L	B	t	a
in. in. in.	in.	ft. in.	in.	in.	in.
5 × 5 × 3	5	3 3	3	5	6½
5 × 5 × 5	5	3 3	5	5¼	7½
6 × 6 × 2	6	3 3	2	5	6¼
6 × 6 × 4	6	3 3	4	5¼	7¼
6 × 6 × 6	6	3 3	6	5½	8¼
7 × 7 × 2	7	3 3	2	5	6½
7 × 7 × 4	7	3 3	4	5¼	7½
7 × 7 × 7	7	3 3	7	5½	9
8 × 8 × 2	8	3 3	2	5	6½
8 × 8 × 4	8	3 3	4	5¼	7½
8 × 8 × 6	8	3 3	6	5½	8½
8 × 8 × 8	8	3 3	8	5½	9½
9 × 9 × 2	9	3 3	2	5	6¾
9 × 9 × 4	9	3 3	4	5¼	7¾
9 × 9 × 6	9	3 3	6	5½	8¾
9 × 9 × 8	9	3 3	8	5½	9¾
10 × 10 × 4	10	3 3	4	5¼	8
10 × 10 × 6	10	3 3	6	5½	9
10 × 10 × 8		3 3	8	5¾	10
11 × 11 × 4	11	3 6	4	5¼	8¼
11 × 11 × 6	11	3 6	6	5½	9¼
11 × 11 × 8	11	3 6	8	5¾	10¼
12 × 12 × 4	12	3 6	4	5¼	8½
12 × 12 × 6	12	3 6	6	5½	9½
12 × 12 × 8	12	3 6	8	5¾	10½
12 × 12 × 12	12	3 6	12	6	13

Standard Weldless Steel Pipes, Spigot and Faucet Ordinary Joint.

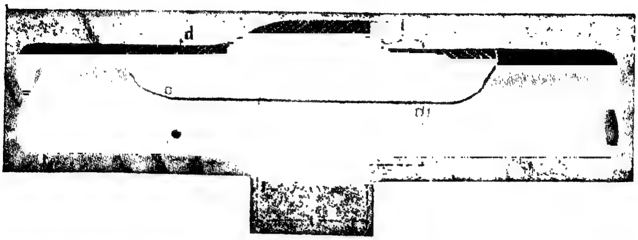


TABLE XI.

Random lengths in ft.	Nominal bore of Tube.	Thickness of Tube I.W.G.	Internal dia. of Socket.	Packing space.	Length of Faucet.	Aprox. weight per foot, without coating and wrapping in lbs.	Aprox. weight per foot including coating and wrapping in lbs.	Weight of Lead Wool in lbs. per joint.	Weight of Hemp yarn in lbs. per joint.	No. of joints per mile.
	D	d	D1	f	t					
18 to 25	1½"	11	2 19/64"	9/32"	3 3/16"	2·13	2·43	·81	·14	230
	2"	11	2 55/64"	5/16"	3 11/32"	2·65	0·03	1·31	·28	230
	2½"	11	3 23/64"	5/16"	3 15/32"	3·33	3·81	1·63	·34	230
18 to 30	3"	10	3⅞"	5/16"	3 37/64"	4·60	5·51	2·12	·37	210
	3½"	9	4 13/32"	5/16"	3 11/16"	5·82	6·45	2·37	·44	210
18 to 35	4"	8, 9	4 59/64"	5/16"	3 13/16"	6·90	7·60	2·63	·50	185
	5"	8	5 15/16"	5/16"	3 15/16"	9·23	10·08	3·19	·56	185
	6"	7	6 31/32"	5/16"	4 1/16"	11·70	12·70	3·75	·69	185
18 to 45	7"	6, 7	8"	5/16"	4 11/64"	14·60	15·75	4·81	·78	175
	8"	5, 6	9 3/64"	5/16"	4 21/64"	18·30	19·60	5·31	1·03	175
	9"	3, 4	10 7/64"	5/16"	4 21/64'	24·10	25·70	6·37	1·12	175
	10"	2, 3	11 7/32"	11/32"	4 7/16"	29·40	31·30	8·00	1·63	175
	12"	1	13¼"	11/32"	4½"	41·60	44·70	9·56	1·87	175

APPENDIX V.

Analysis of Rates for Laying Steel Pipes.

TABLE I.

Item No.	Size of Pipes. Dia in inches.	Length laid in feet.	Labour for excavation and filling in.	Cost per 100 Rft. of excavation and filling in.	Labour for laying.	Cost per 100 Rft. of laying.	Supervision Total.	Supervision per 100 Rft.	Jointing Total.	Jointing per 100 Rft.	Total cost of painting and wrapping.	Cost for painting and wrapping per 100 Rft.	Cost per 100 Rft. laid and jointed complete total of colms. 5, 7, 9, 11 and 13.
1	2	3	4	5	6	7	8	9	10	11	12	13	14
			Rs. A.	Rs. A.	Rs.	Rs. A.	Rs.	R. A. P.	Rs. A.	Rs. A.	Rs.	Rs. A. P.	R. A. P.
(1)	2″	448′	18 6	4 2	14	3 2	2	0 7 0	15 0	3 6	1	0 3 6	11 4 6
(2)	3″	403′	17 8	4 5	13	3 4	2	0 8 0	16 13	4 3	1	0 4 0	19 0 0
(3)	4″	254′	15 12	6 3	10	4 0	2	0 12 9	15 14	6 4	1	0 6 3	17 10 0
(4)	5″	297′	19 4	6 8	12	4 0	2	0 10 9	22 4	7 8	1	0 5 6	19 0 0
(5)	6″	216′	15 12	7 5	12	5 9	2	0 14 9	18 14	8 12	1	0 7 6	23 0 0
(6)	7″	410′	35 0	8 8	27	6 10	4	1 0 0	41 0	10 0	1	0 4 0	26 6 0
(7)	8″	186′	15 12	8 7	13	7 0	2	1 1 0	20 14	11 4	1	0 8 6	28 4 6

APPENDIX V—(Contd.)

Analysis of Rates for Laying Cast Iron Pipes.

TABLE II.

Item No.	Size of Pipes. Dia in inches.	Length laid in feet.	Labour for excavation and filling in.	Cost per 100 Rft. excavation and filling in.	Labour for laying.	Cost per 100 Rft. of laying.	Supervision Total.	Supervision per 100 Rft.	Jointing 25% of total number.	Jointing per 100 Rft.	Rate per 100 Rft. laid and jointed. Complete total of colms. 5, 7, 9 and 11.
1	2	3	4	5	6	7	8	9	10	11	12
			Rs. A.	Rs. A.	Rs. A.	Rs. A.	Rs. A.	Rs. A.	Rs. A.	Rs. A. P.	Rs. A.
(1)	2"	3556'	160 0	4 8	106 0	3 0	31 2	0 14	92 8	2 9 6	11 0
(2)	3"	2097'	94 6	4 8	74 12	3 10	30 3	1 7	43 8	2 1 0	11 10
(3)	4"	1251'	56 5	4 8	47 8	3 13	18 0	1 7	43 12	3 8 0	13 4
(4)	5"	3051'	137 5	4 8	114 10	3 13	43 15	1 7	127 8	4 3 0	13 15
(5)	6"	1147'	51 10	4 8	59 12	5 3	17 14	1 9	63 12	5 9 0	16 13
(6)	7"	4988'	261 14	5 4	196 2	3 15	74 13	1 8	313 0	6 4 0	16 15
(7)	8"	4194'	293 9	7 0	204 7	4 14	68 2	1 10	291 4	6 15 0	20 7
(8)	9"	2574'	180 3	7 0	160 14	6 4	43 7	1 11	214 8	8 5 0	23 4
(9)	10"	794'	55 9	7 0	71 0	8 15	14 14	1 14	77 3	9 12 0	27 9

BRITISH STANDARD FLANGES.

For Working Water Pressure up to 200 lbs. per square inch.

TABLE II.

Size of Valve.	Diameter of Flanges.	Diameter of Bolt Circle.	Thickness of Flange.	Bolts in Flange.	Diameter of Bolts.
Inches.	Inches.	Inches.	Inches.	Number.	Inches.
½	3¾	2⅝	½	4	½
¾	4	2⅞	½	4	½
1	4½	3¼	½	4	½
1¼	4¾	3 7/16	⅝	4	½
1½	5¼	3⅞	⅝	4	½
2	6	4½	¾	4	⅝
2½	6½	5	¾	4	⅝
3	7¼	5¾	¾	4	⅝
3¼	7⅜	6	¾	4	⅝
3½	8	6½	¾	4	⅝
4	8½	7	⅞	4	⅝
4½	9	7½	⅞	8	⅝
5	10	8¼	⅞	8	⅝
5½	11	9¼	⅞	8	⅝
6	11	9¼	⅞	8	⅝
7	12	10¼	1	8	⅝
8	13¼	11½	1	8	⅝
9	14½	12¾	1	8	⅝
10	16	14	1	8	¾
11	17	15	1⅛	8	¾
12	18	16	1⅛	12	¾
13	19¼	17¼	1⅛	12	¾
14	20¾	18½	1¼	12	⅞
15	21¾	19½	1¼	12	⅞
16	22¾	20½	1¼	12	⅞
18	25¼	23	1⅜	12	⅞
20	27¾	25¼	1½	16	⅞
21	29	26½	1½	16	⅞
22	30	27½	1½	16	1
24	32½	29¾	1⅝	16	1

APPENDIX VIII.

Rules for the Management of Water Works in Bengal.

1. The management of the water supply system of every municipality in which such system has been introduced vests in the Municipal Commissioners (hereinafter called "the Commissioners"), and the Commissioners shall perform the duties prescribed in the following rules.

2. The Commissioners are responsible for the proper application of the water supply funds. It shall be their duty to take requisite steps to rectify defects in case they find that water rates are irregularly collected in any instance or are insufficient for carrying out the purposes of Part VII of the Bengal Municipal Act, 1884.

3. The Commissioners shall frame the annual budget of income and expenditure on account of the water-works, and shall submit it to the Chief Engineer, Public Health Department (hereinafter called "the Chief Engineer"), for any remarks he may consider necessary. The Chief Engineer shall return the budget with his remarks to the Chairman of the Municipality.

4. The Municipality shall maintain a separate account of the water supply fund and the Chairman shall, at the end of each month, prepare a statement of the accounts of this fund, which he shall submit to the Commissioners at their next meeting.

5. The Commissioners at a meeting shall, every month, inquire into all matters connected with the water supply of the municipality and examine and pass the accounts submitted under rule 4, and shall record that the accounts have been passed in the book of the minutes of the proceedings of the meeting.

6. In all engineering matters connected with the water-works the Commissioners shall be guided by the advice of the

Chief Engineer. The Commissioners shall afford the Chief Engineer all information he may from time to time require, and shall consider and attend to all communications received from him.

7. The Chief Engineer, and the inspecting officers of the Engineering Branch of the Public Health Department may, whenever it is desirable, correspond direct with the Superintendent of the Pumping Station in regard to technical matters. Copies of all such letters shall be forwarded to the Chairman of the Municipality.

8. (*1*) An officer of the Engineering Branch, Public Health Department, shall visit the water-works not less than once a year, and his report thereon shall be submitted to the Sanitary Board by the Chief Engineer, a copy being forwarded to the Municipal Commissioners. The Commissioners shall arrange for the proper inspection of the boilers and pumping machinery not less than twice a year by a qualified Machinery Inspector or firm of Machinery Inspectors who must be approved by the Chief Engineer.

(*2*) The reports of the Inspectors must be submitted,—

(*i*) in the case of boilers and pumping machinery worked by steam, in P. H. D. Form No. 6 (Appendix A),

(*ii*) in the case of engines and pumps using oil fuel, in P. H. D. Form No. 6A (Appendix A),

in the case of mechanical fitters, in P. H. D. Form No. 6B (Appendix A), to the Chief Engineer:

Provided that in the case of small water-works the said inspection may, with the consent of the Chief Engineer, be held only once a year.

9. Until all points raised in the reports of the Chief Engineer and the Machinery Inspector have been disposed of, the Commissioners shall submit to the District Magistrate, for transmission to the Chief Engineer, a monthly progress report showing how far effect has been given to the recommendations made in the reports.

10. (*1*) The Commissioners shall cause to be kept at the Pumping Station—

(*a*) a stock account in P. H. D. Form No. 4 (Appendix A), showing the daily transactions in coal and engine room stores ;

(*b*) an engine room log in the P. H. D. Form No. 3 (*a*) or 3 (*b*) (Appendix A) ;

(*c*) a workshop log in P. H. D. Form No. 3 (*c*) (Appendix A) ;

(*d*) a filter-bed log in P. H. D. Form No. 5 (Appendix A) ; or a filter log in P. H. D. Form No. 5A (Appendix A) ;

(*e*) a settling tank log in P. H. D. Form No. 5B ; and

(*f*) such other forms as the Chief Engineer may, from time to time, prescribe.

(*2*) The said forms shall be written up daily by the Superintendent of the Pumping Station and shall be available at all times for inspecting officers and visitors appointed under clause (*1*) of rule 15.

11. The Commissioners shall submit to the Chief Engineer not later than the tenth of each month :—

(*a*) indicator diagrams from each engine in P. H. D. Form No. 1 (Appendix A) for one day in the preceding month, which must be accompanied by a copy of the engine room log, Form No. 3 (Appendix A), for the day on which they are taken ;

(*b*) an abstract of work done by the pumping machinery during the preceding month in P. H. D. Form No. 2 (Appendix A) ;

(*c*) a copy of the filter bed log or filter log, and settling tank log, P. H. D. Forms Nos. 5, 5A and 5B (Appendix A), for the same day of the preceding month as that to which the indicator diagrams, referred to in clause (*a*), apply.

12. The Commissioners shall submit quarterly to the Chief Engineer a return showing the number of metered and unmetered house connections, and the quantity of water measured by meter and charged for in P. H. D. Form No. 7 (Appendix A).

13. The Commissioners shall in addition furnish such other information as the Chief Engineer may, from time to time, require.

14. It shall be the duty of the Commissioners to see that the rules for working settling tanks and slow sand and mechanical filters in water-works (Appendix B) are properly attended to by the Superintendent of the Pumping Station. If, owing to the design of the works, the rules cannot be conformed to, the matter shall be referred to the Chief Engineer for orders, which will thus take, in these particular matters, the place of the rules in Appendix B.

15. (*1*) The Commissioners shall appoint two of their numbers in rotation to be monthly visitors of the water-works.

(*2*) It shall be the duty of these visitors to ascertain by enquiry—

(*a*) whether an efficient supply of water is being given ;

(*b*) whether the Superintendent of the Pumping Station and the water-works establishment are attending to their duties and a proper discipline is being maintained ;

(*c*) whether the works, machinery, plant, implements, materials and all other things connected with the water-works, are being properly maintained in an efficient condition ;

(*d*) whether these rules are being attended to in all respects ; and

(*e*) whether any other matters in connection with the water-works require attention.

(*3*) The visitors shall report in writing to the Commissioners in meeting the result of their inspection, and a copy of their report, with a copy of any resolution that may be passed thereon by the Commissioners, shall be sent to the Chief Engineer.

16. A visitor's book shall be kept at the pumping station in which the visitor's remarks shall be recorded.

17. An extract of the proceedings relating to the water supply system at each meeting of the Commissioners shall be forwarded, within a week of the date of the meeting, to the Chief Engineer, who may, if he thinks fit, and shall, if the Commissioners so desire, submit it with his remarks to the Sanitary Board.

18. The chemical and bacteriological analysis of water from each water-works shall be carried out in the Public Health Laboratory monthly, and as often as the Director of the Public Health Laboratory shall consider necessary, and the results of the analysis shall be communicated by him to the municipality and shall be laid before the Commissioners at a meeting.

But the analysis of water from the water-works of the Darjeeling Municipality may be carried out at its own laboratory.

19. The Director of the Public Health Laboratory after an inspection of any water-works by himself or any member of his staff shall forward a copy of the inspection report to the Chief Engineer.

APPENDIX A.

[*See* Rule 11 (*a*).]

P.H.D. Form No. 1.

Details of Diagram.

Name of station ....

Date when taken ....

Distinguishing letter of Engine ....

Cut off ....

Diameter of Cylinder ....

Diameter of Piston Rod ....

Net area of Piston ....

Length of Stroke ....

Revolutions per minute ....

Top or bottom ....

Pressure by Steam Gange ....

Temperature of Condenser ....

Vaccum on Gange ....

Pressure on Pumps ....

Indicated HP High ....

,, ,, Intermediate ....

,, ,, Low ....

Total indicated HP ....

Consumption of Coal per hour ....

,, ,, per indicated HP per hour ....

Name of Coal used ....

Quality of Coal used ....

Scale of indicator ....

APPENDIX A.

P.H.D. Form No. 2.

[*See* Rule 11 (*b*).]

__________________**Water-works.**

*Abstract of work done by Pumping Machinery for the month of*__________*19* .

	Number of hours pumps at work.	Average lift, including suction and friction.	Total quantity of water pumped in gallons.	Total work done in foot pounds.	Number of p. h. p. hours.	Total quantity and name of coal in cwts. or total quantity and description of fuel oil in lbs.	Consumption of coal or oil in lbs. per p. h. p.	Remarks.
Unfiltered water pumps								
Filtered water pumps								

Superintendent.

*Dated*______________*19*

APPENDIX A.

P.H.D. Form No. 3 (*a*).

[*See Rules* 10 (*1*) (*b*) and 11 (*a*).]

————————————— **Water-works.**

Engine log for the——————*of*——————*19* .

Engine at work.	Hour worked.		Counter when engine started.	Counter when engine stopped.	Total revolutions made.	Pressure on pumps.			Suction lift.			Total average lift.	Total gallon pumped.	Boilers at work.			Coal in cwts.		Remarks.
	From	To				Hour.	Feet.	Average.	Hour.	Feet.	Average.			1.	2.	3.	At work.	Banking and geting up steam.	

Superintendent.

Dated——————*19* .

APPENDIX A.

P. H. D. Form No. 3 (*b*).

————— Water-works.

Engine log for the————of————19

	Engines.				Pumps									
	Hours worked.				Pressure.			Suction lift.						
	From	To	Average revolution per minute.	Total consumption of fuel in lbs.	Hour.	Feet.	Average.	Hour.	Feet.	Average.	Total average lift.	Average revolution per minute.	Total gallon pumped.	Remarks.
Engine at work ..														

Superintendent.

Dated————19 .

APPENDIX A.

P. H. D. Form No. 3 (*c*).

————— Water-works.

Workshop Log for the month of————19

Date.	Description of work.	Number of hours worked by workshop engine.	Labour employed.	Fuel consumed.	Stores used.	Remarks.
1	2	3	4	5	6	7

Dated————19 . *Superintendent.*

P. H. D. Form No. 4.

APPENDIX A.

[*See* Rule 10 (1) (*a*).]

____________________ **Water-works.**

Return of Coal and Stores consumed for the month of __________ 19 .

Date of month.	Coal in Maunds.		Fuel Oil in Gallons.		Castor Oil is Seers.		Lubricating oil.		Cyllnder Oil.		Tallow.		Jute.		Cotton. waste.		Alum in lbs.		Turpentine in Pints.		Number of hours engine worked.				Remarks.
	Received.	Consumed.	Received.	Consumed.	Received.	Consumed.	Received.	Consumed.	Received.	Consumed.	Received.	Consumed.	Received.	Consumed.	Received.	Consumed.	Received,	Consumed.	Received.	Consumed.	Horse-power.	Horse-power.	Horse-power.	Horse-power.	
Quantity in Stock at commencement of month.																									
1																									
2																									
3																									
4																									
5																									
6																									
...																									
...																									
30																									
31																									
Total ..																									
Quantity in Stock at end of month.																									

Superintendent.

P. H. D. Form No. 5.

APPENDIX A.

[*See* Rules 10 (*1*) (*c*) and 11 (*c*).]

————————Water-works.

Filter log for the————————of————————19

SLOW SAND FILTERS.

Number of filter-bed.	Hour.	Quantity of water filtered.	Rate of filtration.	Depth of fine sand.*	Level of water in filter-bed.	Level of water in filter-well.	Filtration head.	Remarks.
I	3 A.M. 6 ,, 9 ,, 12 Midday. 3 P.M. 6 ,, 9 ,, 12 Midnight.							
II	3 A.M. 6 ,, 9 ,, 12 Midday. 3 P.M. 6 ,, 9 ,, 12 Midnight.							

NOTE.—All gauges should be read every three hours; all levels to be referred to the bottom of the filter-bed.

In the column of remarks should be entered an account of all work, etc., done on the filter-beds such as scraping or renewal of filtering materials.

* This column is for reference only and should be entered up once a day. The depth given is that ascertained on the occasion of the last scraping.

Superintendent.

Dated————————192

P. H. D. Form No. 5A.

APPENDIX A.

[*See* Rules 10 (*1*) (*c*) and 11 (*c*).]

——————Water-works.

Filter log for the—————— *of*—————— *19*

MECHANICAL FILTERS.

Hour.	Total quantity of water filtered.	Quantity of alum used.	Filters in use.									Remarks.
			No. 1.			No. 2.			No. 3.			
			Filtration head.	Hour and length of time of washing.	Quantity of wash water used in gallons.	Filtration head.	Hour and length of time of washing.	Quantity of wash water used in gallons.	Filtration head.	Hour and length of time of washing.	Quantity of wash water used in gallons.	
3 A.M.												
6 "												
9 "												
12 Midday.												
3 P.M.												
6 "												
9 "												
12 Midnight.												

NOTE.—The total quantity of water filtered and of alum used need only be entered for the 24 hours.

Superintendent.

Dated——————*192* .

P. H. D. Form No. 5B.

APPENDIX A.

[*See* Rules 10 (*1*) (*c*) and 11 (*c*).]

——————**Water-works.**

Settling Tank Log for the—————— *of*—————— *19*

No. of settling Tank.	Hour.	Depth of water in tank. (Feet.)*	Quantity of unfiltered water pumped in during the preceding 24 hours. (Gallons.)*	Quantity of water preceding 24 hours. (Gallons.)*	Quantity of alum added drawn off during the during the preceding 24 hours. (lbs.)*	Remarks.
I. (Capacity ....gallons.)	6 A.M. 6 P.M.					
II. (Capacity ....gallons.)	6 A.M. 6 P.M.					

* To be entered up at 6 P.M. daily.

Superintendent.

Dated—————————*19* .

P. H. D. Form No. 6.

Report on an examination of the Boilers, Engines and Pumps of the————————————————— Water-works made by ——————————————————————— on the ———————————————————————

BOILERS.

1. Description of boilers, giving makers' name and date of erection. Give grate area and calculated horse-power.	
2. When and by whom last examined :— (*a*) If examined by an Inspector of Steam-boilers under Act III (B.C.) of 1879, give name of Inspector and number and date of last certificate. (*b*) Note working pressures previous to last examination ; and, if any reduction in pressure was made by the Inspector, state how much, and why such reduction was made.	

Notes—If the boilers are working under certificates granted by an Inspector under Act III (B.C.) of 1879, questions 3, 4, 5, and 6 need not be answered.

3. Have you examined the boilers internally and externally ; if so, with what results? Give thickness of scale, if any, and state whether you had it removed.	

BOILERS—*contd.*

4. Did you test the boilers by hydraulic pressure ; if so, up to what pressure? Did you ascertain that the steam-gauge was correct, and that the steam relief-valve was in working order, and not over-weighted, before applying the hydraulic test?	
5. What working pressures do you now recommend?	
6. Have you examined all the boiler-fittings, such as safety-valves, feed water-pipe, blow-off cocks, steam and water-guages, etc.? State if they are all in good working order, and if not, what is required to make them so? *Note—The safety-valves should not be weighted to more than 10 lbs. (preferably 5 lbs.) above the working pressure.*	
7. Are the boilers blown out regularly, and safety-valves lifted to ensure their not sticking ; and is a record kept of the dates on which this has been done since that last inspection?	

BOILERS—*contd.*

8. Is the floor of the boiler-house kept dry and in good order? (*a*) Where are the ashes slaked? (*b*) When was soot last removed from the flues? (*c*) Are the flues free from moisture during the rainy season?	
9. State which of the follow are available for filling the boiler, and which is generally used— (*a*) Feed-pump on engine. (*b*) Donkey-pump. (*c*) Injector. (*d*) Cold water pressure from the mains.	
10. State average fuel consumption since last report giving the percentage of ashes and kind of fuel used. If coal be used, give name of colliery from whence obtained.	
11. General remarks. *Note.—Any repairs that have been done in the boiler-house since last inspection should be recorded here.*	

ENGINES.

12. Description of Engines, noting also makers' name, date of erection, diameters of cylinders, and length of stroke.	
13. When and by whom last examined?	
14. Did you examine the interiors of the cylinders, and if so, with what result?	
15. Did you examine the steam-valves, and if so, with what result?	
16. Did you take any indicator diagrams? If so, attach to this report a set worked out, with full particulars noted:— (*a*) State whether you consider the valves are properly set for the most economical working of the engine. (*b*) State whether you consider the indicator diagrams are satisfactory or not, and whether any difference in them is apparent. If so, what, in your opinion, has caused the difference. (*c*) State steam consumption as ascertained from the diagrams.	
17. Are all stuffing boxes and glands kept properly packed, and steam-pipes free from leaks?	

ENGINES—*contd.*

18. What vacuum is generally maintained?	
19. Is the air-pump in good order? Give temperature of its discharges. *Note—The temperature should not exceed 115°.*	
20. Are the lubricants in use of good and suitable quality, and is sufficiently large supply of all stores kept in hand?	
21. General remarks. *Note—All repairs, however slight, that have been carried out since last report, should be mentioned here.*	

PUMPS.

22. Description of pumps, noting also diameters of buckets or plungers, length of stroke, number and size of valves.	
23. Did you examine all buckets and plungers, and if so, with what result?	

PUMPS—*contd.*

24. What do you consider is the percentage of "slip"? (a) What do you find the mechanical efficiency of the engines?	
25. Were the pumps working smoothly, evenly and without noise or banging of valves?	
26. Are the air-vessels kept properly charged with air? (a) State means of doing so.	
27. General remarks. *Note—All repairs that have been carried out since last report should be mentioned here.*	

GENERAL.

28. Is the staff at the pumping station sufficient and the health of the employees generally good?	
29. General remarks.	

Signed ________________

Date ________________ *Rank* ________________

P. H. D. Form No. 6A.

Report on an examination of the Oil Engines and Pumps of the water-works, made by Mr.——————————
on the——————————

ENGINES.

1. Description of engines, noting also makers' name, date of erection, number and diameter of cylinders, length of stroke, type of engine, viz., two or four stroke, and the average revolutions per minute at full load.	
2. When and by whom last examined?	
3. Did you examine the interiors of the cylinders, and, if so, with what result?	
4. Did you examine the suction, exhaust, fuel and starting valves, and also the fuel pumps, and, if so, with what result?	
5. Did you examine the cam shaft, cams and gearing? If so, with what result?	
6. Did you examine the crankshaft bearings and also the big end and small end bearings of connecting rod, and, if, so, with what result?	

ENGINES—(*contd.*)

7. Did you take any indicator diagrams? If so, attach to this report a set worked out, with full particulars noted:— (*a*) State whether you consider the indicator diagrams are satisfactory or not, and whether any difference in them is apparent. If so, what in your opinion, has caused the difference.	
8. Are all stuffing-boxes and glands kept properly packed, and pipes free from leaks?	
9. Give temperature of the water in the cooling tanks. Do you consider the capacity of the tanks sufficient and the arrangement of the water connects satisfactory? NOTE.—The temperature of cooling water at outlet should not exceed 130°F.	
10. Are the lubricants in use of good and suitable quality, and is a sufficiently large supply of all stores kept in hand?	
11. State average fuel consumption per P.H.P. per hour and whether crude or refined oil is used.	

ENGINES—*contd.*

12. Are the engines fitted with air compressors? If so, what type? Did you examine interiors of the cylinders, also the valves, and, if so, with what results?	
13. General remarks. NOTE.—All repairs, however slight that have been carried out since last report, should be mentioned here.	

PUMPS.

14. Description of pumps. If plunger or bucket type, give diameter and number of same and stroke; also number and size of valves. If centrifugal type, give size of pump.	
15. Did you examine all buckets and plungers and the interior of the pumps; if so, with what result?	
16. What do you consider is the percentage of "slip". What do you find the overall mechanical efficiency of the pumps and engines?	
17. Were the pumps working smoothly, evenly, and without noise, or banging of valves?	

PUMPS—(*contd.*)

18. Are the air-vessels kept properly charged with air? (*a*) State means of doing so.	
19. General remarks. NOTE.—All repairs that have been carried out since last report should be mentioned here.	

GENERAL.

20. Is the staff at the Pumping-station sufficient, and the health of the employees generally good?	
21. General remarks.	

Dated——————*19* .

Signed ——————

Rank——————

P. H. D. Form No. 6B.

Report on an Examination of Mechanical Filters of the —————— *Water-works made by Mr.*——————
on the——————

MECHANICAL FILTERS.

1. Description and type.	
2. If Gravity Filter, state— (*a*) Condition of agitator. (*b*) Condition of controller. (*c*) Condition of alumina apparatus.	

MECHANICAL FILTERS—*contd.*

3. If Pressure Filter, state— (*a*) Condition of brass pipes on coagulant pot. (*b*) Condition of all exterior sluice valves after testing through waste valve. (*c*) Condition of main operating valve.	
4. (*a*) State level of filter bed below top of filter shell. (*b*) What quantity of alumina is being added daily? (*c*) Is alumina being added in one or more doses? (*d*) How long settlement is allowed after addition of almina?	
5. Did you see the filter washed?	
6. (*a*) How often is the filter washed? (*b*) What quantity of filtered water is used for washing and running to waste?	
7. State what quantity of water is filtered daily.	
8. REMARKS.	

Dated——————————*19* .

Signed——————————

Rank——————————

P. H. D. Form No. 7.

APPENDIX A.

—————————— **Water-works.**

Quarterly return of Public and Private metered and connections.

Ward No.	Street standposts.		Ground hydrants.		House connections.		Number of meters repaired during quarter.	Total consumption of water in metered connection.	Total excess consumption charged for.	Amount charged.	Remarks.
	No.	Number repaired during quarter.	No.	Number repaired during quarter.	Metered number.	Unmetered number.					
1	2	3	4	5	6	7	8	9	10	11	12

Dated——————*19*　　　　*Superintendent.*

APPENDIX B.

(See Rule 14.)

RULES FOR WORKING SETTLING TANKS AND SLOW SAND AND MECHANICAL FILTERS IN WATER-WORKS IN BENGAL.

Settling Tanks.

1. It is important that water drawn from the rivers of Bengal should have as long a settlement as possible before being passed on to slow sand filters, in order that the action of sunlight, the precipitation of suspended matter and the natural tendency to elimination of pathogenic bacteria may have their maximum purifying effect.

2. Settling tanks are worked either on (*a*) the continuous flow, or (*b*) the intermittent system. In the first system, the

tanks are kept full, and the water is continually admitted at one end and drawn off from near the surface at the other. In order that this system may be properly used with slow sand filters, the unfiltered water pumps must be worked continuously throughout the 24 hours or the settling tanks must be fed from a storage reservoir. When designed on the intermittent system, each tank in turn is filled and then kept full until the tanks filled before it are drawn down; the settled water is then decanted through a floating arm or similar apparatus until the tank is lowered to the lowest draw off level, when it is again filled.

The whole available storage capacity of settling tanks should always be used as far as possible. On the other hand care must be taken in tanks unprovided with fixed overflows not to fill them to a greater depth than they are designed for.

3. When a deposit of from 18 inches to 2 feet of silt has formed at the bottom, the tank should be emptied and cleansed out. This work must always be done in the dry weather and preferably at the end of October or as soon after as possible.

4. In the rains and when there is an excessive amount of suspended matter in the raw water, clarification will generally have to be assisted by the use of a coagulant. The precipitant most commonly used in Bengal water-works is aluminoferric. This depends for its effective action upon the temporary hardness of the water. If more of the chemical is used than can be decomposed by the carbonates present, it will remain dissolved in the water and will be wasted.

Generally speaking, not more than two grains of aluminoferric are required per gallon of water. Aluminoferric and sulphate of aluminum should always be ordered under a proper specification. Samples of each consignment of the precipitant should be sent to the Director of Public Health Laboratories for analysis to see if the samples conform to the specification. His advise will also be necessary from time to time as to the quantity of precipitant to be added to the water.

5. The aluminoferric must be added at the inlet end of the settling tanks. A solution of a fixed strength should be

prepared and the rate of flow of the solution ascertained and regulated by experiment in cases where there is no automatic apparatus for its addition. The method of adding the salt in solid form by suspending in a basket is unsatisfactory and should not be adopted.

Slow sand filters.

5. The rate of delivery on to a sand filter is regulated by (*a*) the available head between the settling tanks and the inlet valve and (*b*) the amount the inlet valve is open.

The rate of flow through the filter depends upon (*c*) the resistance of the sand in the filter. In order to overcome this resistance a difference in level forms between the surface of water on the filter-bed and the water in the outlet well. This is called the filtration head. The longer the filter-bed is in operation, the greater becomes the resistance of the sand and consequently a larger filtration head is necessary to force the water through the filter.

The discharge from the outlet well to the clear water reservoir depends on (*d*) the amount the outlet valve is open or in the case of a telescopic outlet the extent to which the bell-mouth is lowered.

It is obvious that these factors are mutually interdependent, for more water cannot flow out of a filter than is delivered on to it, nor than can make its way through the sand.

Factors (*a*) and (*c*) continually tend to vary, so (*b*) and (*d*) must also be altered from time to time in order to maintain a constant flow through the filter. In some filters the inlet is controlled by a ball valve which keeps the level of the water on the filter always the same. In this case, the flow through the filter is regulated by (*d*). In all filters the surface of the water on the sand should be always kept as nearly at same levels as possible and filtration head regulated by the outlet valve or telescopic weir. The depth of water on the sand should not be less than 2 feet. Gauges measuring to the same datum should be fixed on the wall of the filter and in the outlet well so that the difference in level can be easily measured.

7. Filtration should be carried on continuously and as nearly as possible at the same rate, which should be between 3 or 4 inches vertical an hour. The rate of flow must on no account exceed the latter figure.

Where there is no direct method of measuring the flow of water through a filter such as an outlet gauge notch or a meter, it may be ascertained by closing the inlet valve and leaving the outlet valve open. The fall of the surface of the water in inches at the end of an hour will give the rate of flow per hour.

8. In a new filter or one recently scraped, the filtration head required will be very small but its increase will be rapid. In the usual way, a filter should be scraped when the filtration head reaches 18 inches. In certain cases it may be safe to exceed this limit, but before allowing a greater working head the advice of the Chief Engineer should be obtained.

Filters should not be scraped until it is necessary, for not only does frequent scraping increase the cost of operation but for some time after scraping the efficiency of the filters is reduced.

9. A filter must be scraped by carefully removing the slimy ooze which has formed on the top, together with about ¼ inch of sand. The sand removed may subsequently be used again if properly washed in a sand-washer. In the usual way when the depth of fine sand is reduced to 18 inches it is time to replenish the filter. Before replenishing about two or three inches of sand is to be removed, clean washed sand is then to be added until the original thickness is obtained.

10. If the water filtered has been of a very bad quality or the filter has been mismanaged, or been long in use the whole filter may require to be renewed. In such case the advice of the Chief Engineer, Public Health Department, must be obtained, when full instructions as to the work required will be issued.

11. After a filter-bed has been scraped, replenished or renewed it should, if possible, be charged from below by admitting filtered water very slowly until the surface is about three inches above the sand ; the filling can then be completed

from above. If there is no arrangement for filling from below, the filling must be very carefully and slowly done from above.

After filling, water should be passed through the filter commencing at the rate not exceeding one inch vertical per hour and run to waste for 24 hours after scraping and 48 hours after replenishing. The rate of filtration should be increased gradually after putting the filter into use. If possible the maximum rate of flow should not be reached for four or five days. After complete renewal the filter must not be put into operation until the Director of Public Health Laboratories has analysed samples of the water and pronounced them safe.

12. Daily records must kept in the filter bed log book [P. H. D. Form No. 5 (Appendix A)] of the quantity of water filtered and the filtration head recorded at intervals of three hours throughout the day.

Mechanical filters.

13. For each installation of mechanical filters, rules for working must be obtained from the makers of the plant, but the following points must be attended to in all cases:—

(*i*) The efficiency of the plant depends upon the proper proportioning and correct addition of the coagulant. The actual amount of coagulant required depends on the design of the installation, the type of filter, the quality and hardness of the water and general local conditions. It will vary during different season of the year and can be best ascertained by experiment in actual working. Such experiments require scientific supervision and must be carried out under the instructions of the Chief Engineer, Public Health Department, who will when necessary take the advice of the Director of Public Health Laboratories on the subject.

(*ii*) The precipitant generally used is sulphate of alumina and this should be purchased under a proper specification of chemical composition. The precipitant is made up into a strong solution in large vats and added to the raw water by an adjustable apparatus.

The water so treated is then allowed to settle for sufficient time to allow the formation of optimism flock.

In cases where sedimentation continues for a considerable time after the coagulant has been applied, it may be necessary to add a small additional quantity of the chemical to the water immediately before passing it to the filters.

In pressure filters if sulphate of alumina is allowed to stand in the alum pots for any length of time there is a tendency for it to deposit and clog the small pipe leading to the filter inlet. In such cases the use of crystal potash alumina is recommended. If sulphate of alumina is used it is necessary to keep the solution stirred and also to examine the alum pipes to see that the flow is taking place properly. In any properly designed installation means are provided for testing the rate of flow through the alum pipe at any time.

Aluminoferric and ammonia alum are not to be used in mechanical filters.

(*iii*) The rate of filtration should be kept as nearly as possible constant and must not exceed the rate for which the filters had been designed. All filters used on the gravity system are provided with automatic controllers, and these must on no account be tampered with or made to pass more water than intended.

(*iv*) The maximum filtration head permissible varies with different types of plant ; it is usually not more than 10 feet. When the maximum head is reached, as shown by the indicator, the filter must be washed.

(*v*) Washing must always be done with filtered water. In the case of a battery of two or more pressure filters, they may be arranged so that one can be washed with filtered water from another. Washing must be continued until the waste water becomes quite clear. After washing, the filtered water must be run to waste for 15 minutes before connecting to the town. All valves operated during the process of washing must be opened and closed slowly.

(*vi*) The addition of coagulant throughout the filtering process is unnecessary. After four hours of filtration with coagulant a sufficiently impervious layer is formed on and in the upper layers of sand to stop the passage of bacteria and filtration can then be carried on for two days or more without the addition of any more alum.

(*vii*) Daily records must be kept in the filter log book [P. H. D. Form No. 5A (Appendix A)] of the quantity of water filtered, the amount of sulphate of alumina used, the hours during which filtration has been carried on the filtration head (recorded at intervals of three hours), the length of time taken to wash each filter, and the hour at which it was washed.

APPENDIX IX.
STATISTICS OF WATERWORKS IN BENGAL.

Serial No.	Name of Municipaltiy.	Area served in sq. miles.	Population.	Average daily supply in gallons.	Supply per head of population per day in gallons.	Total capital cost.	Capital cost per gallon per day.	Capital cost per head of popula-tion.
						Rs.	Rs.	Rs.
1	Berhampur	6·6	26,670	3,13,428	11·75	3,79,971	1·212	14·25
2	Burdwan	3·3	49,200	4,47,419	9·09	5,75,896	1·28	11·70
3	Dacca	7·0	1,17,900	20,19,938	17·13	16,33,854	0·79	13·86
4	Howrah	10·10	1,95,300	39,79,642	20·38	39,52,407	0·99	20·24
5	Mymensingh	2·50	25,200	3,32,113	13·18	4,62,131	1·38	18·34
6	Khulna	2·00	10,000	80,792	8·08	1,19,629	1·48	11·96
7	Jessore	4·00	8,000	36,767	4·60	1,96,874	5·26	24·61
8	Hooghly Chinsurah ...	5·00	29,900	4,61,272	15·40	6,22,095	1·34	20·78
9	Serampur	1·62	33,200	2,84,833	8·58	3,19,363	1·12	9·62
10	Bankura	2·50	22,273	1,28,257	5·75	1,18,229	0·92	5·31
11	Chittagong	4·25	40,000	4,61,583	11·54	3,96,587	0·88	9·91
12	Utterpara	0·75	9,394	1,06,640	11·35	1,93,618	1·89	20·61
13	Midnapur	3·50	30,000	3,72,229	12·40	5,55,810	1·48	18·53
14	Krishnagar	7·00	22,300	1,69,457	7·60	4,12,595	2·43	18·05
15	Faridpur	5·00	14,000	56,834	4·06	1,50,417	2·64	10·74
16	Raneegunj	1·85	16,349	1,28,318	7·85	3,26,935	2·54	20·00
17	Natore	1·60	7,000	18,914	2·70	1,04,492	5·54	14·93
18	Suri	3·00	10,900	50,741	4·65	2,32,612	4·58	21·34
19	Naraingunj	3·00	30,600	3,26,152	10·66	2,53,312	0·78	8·27

APPENDIX X.

ELIMINATION OF TASTES AND ODOURS.

During the last decade much attention has been paid to the elimination and prevention of tastes and odours in public water supplies in Europe and America. This is due to the production of unpleasant and objectionable characteristics in water owing to the increased use of chlorine for disinfection and also to the pouring of larger volume of industrial wastes into sources of supply. The problem of tastes and odours removal is not new, it existed before, many surface waters possesses natural odours which are often disagreeable and give cause of complaint.

Whipple classifies odours as follows:—

(*i*) Odours caused by organic matters other than living organisms.

(*ii*) Odours caused by living organisms.

(*iii*) Odours caused by chlorine or chlorine compounds and by the action of chlorine upon any of the odour producing agencies.

As to the character of tastes and odours there is the difficulty of finding exact expression for each kind of taste met with as the sense of smell or taste of different person is different. In practice, it is customary to distinguish the taste and odour with the taste and odour of some object of which man is most familiar. The odours caused by organic matters present in water is generally represented as *Straw-like, Swamp-like* or *Peaty,* while those produced by microscopical growths are classified as *Algæ, Aromatic* or *Fishy* etc. The odours caused by chlorination may be due to free chlorine present in water or to chlorine compounds formed. In the former case the taste is described as *Chlorinous,* and in the latter case, a most disagreeable taste

is generally produced by the combination of chlorine with phenolic substances carried by wastes from idustries and is generally called *Iodoform* or *Medicinal taste.* Chlorine acting upon algæ and other organisms present in water also produce disagreeable odour.

Rapid progress has been made within last few years in the development of methods of elimination of tastes and odours. Of the several methods in use, the most widely adopted are:—

(*i*) Chloramine treatment.

(*ii*) Activated Carbon treatment.

We have not got any experience with either of these methods in this country and the description given in the following pages is of the practice followed in Europe and America.

Chloramine Treatment—Chloramines are produced in water by the combination of chlorine with ammonia or some ammonium salts present in water. This was reported by Rideal in 1910 while treating sewage with chlorine. Very little is known as to the physical and chemical properties of chloramine. All that is known that they are oily, extremely unstable and soluble in water. There are two kinds of chloramine—(i) *Mono-chloramine* and (ii) *Di-chloramine.* These are produced by the successive replacement of hydrogen in ammonia by chlorine and can be obtained by mixing chlorine with ammonia or ammonium salts. Writing about the inter-relation of ammonia with chlorine Ellms in a recent paper writes as follows:—"The chemistry of the reactions involved when ammonia is applied to a natural water prior to chlorination is more or less obscure. It seems probable, however, that the formation of monochloro-amine and dichloro-amine are dependent upon the hydrogen-ion concentration of the water being treated. Probably in naturally alkaline waters, a mixture of these two amines results. The combination of chlorine or hypochlorous ions with phenolic compounds, when the latter are present, to form chlor-phenolic bodies producing foul tastes, is apparently prevented by the presence of ammonia.

The theoretical ratio of NH_3 (ammonia) to Cl (chlorine)

to form monochloro-amine is 1 to 4·2 ; and for the dichloro-amine it is 1 to 8·3. Under the acid conditions a theoretical ratio of 1 to 3·12 produces NCl_3 ; and under alkaline conditions a theoretical ratio of 1 to 1·55 forms nitrogen gas. It is, therefore, probable that too great an excess of ammonia is liable to prevent the formation of chloro-amines, while an insufficient quantity may result in an excess of chlorine with the consequent formation of chlor-phenolic taste producing compounds, should phenols be present. Within the range of pH values for most water supplies, there is probably formed a mixture of monochloro- and dichloro-amines.

The disinfecting property of the chloro-amines is well established, and it is, therefore, desirable to provide if possible the optimum conditions for their production when depending upon them for bactericidal action. They have a somewhat slower action than does chlorine used alone, but they have a continuing inhibitory effect upon bacterial growths which is of great value for maintaining a water supply in good condition during its passage through pipe lines and reservoirs to the consumers. These facts have been confirmed in our experimental work."

Regarding relative bactericidal efficiency of chloro-amine and chlorine Enslow states, "Regardless of the form of active chlorine applied—whether it be as chlorine, hypochlorite or chloramine—the *rate* of destruction of bacteria is radically affected by changing the pH value of the medium. If rapid disinfection is requisite, it will be necessary to maintain relatively higher residual chlorine concentration in waters of high pH values. Above pH 8·0 the rate of bacterial destruction in water may be materially retarded for a given residual chlorine value whereas lowering the PH value under 7·0 increases the efficacy of a small residual chlorine content very markedly. Holwerda calls attention to the observation that 0·02 p.p.m. residual chlorine (ortho-tolidin test) at pH 5·9 is practically as effective as ten times that concentration (0·2 p.p.m.) at pH 9·0. An example of the slowing down of disinfection with increasing

pH values, even with relatively high residual chlorine content (0·5 p.p.m.) follows:—

	pH	p.p.m. Residual Cl.	Minutes required for disinfection.
(a) ...	4·5	0·5	20
(b) ...	6·8	0·5	30
(c) ...	8·5	0·5	60

According to Chapin the following ratios of chloramines must exist at the pH values shown above:—

In (a) 100%—dichloramine ($NHCl_2$)

In (b) { 50%—dichloramine
50%—monochloramine (NH_2Cl).

In (c) 100%—monochloramine.

The residual chlorine concentration in the chloramine form which is required to produce efficient disinfection varies with temperature rather markedly. Even a change of 12 deg. C. representing a drop from 27° to 15°C. apparently has a marked effect on the chlorine residual required. Likewise low temperatures materially retard the rate of color development in the orthotolodin test.

Holwerda considers any claims that the disinfection powers of chloramine chlorine are greater than ordinary chlorine are in error. It may be *apparently greater* because of the slower absorption of the chloramine chlorine by organic substances but is *actually less powerful.* If compared on the basis of residual chlorine concentrations chlorine applied as hypochlorite or in the form of chlorine water is more powerful and considerably more rapid than chloramine in effects produced. Chloramine chlorine has one advantage in that it remains unabsorbed for a much longer period and therefore has a special value in certain cases. To overcome the disadvantages from chloramine production it will be necessary to weigh carefully the effects of alkalinities in excess of pH 7 and lowering of temperatures. The residual chlorine content should be increased under such circumstances to values in excess of those formerly required when chlorine alone was in use."

Elaborate experiments were also carried out by Gerstien in collaboration with Baylis and others and the conclusions of their results are reported as follows:—

1. The treatment is effective in preventing the formation of the chlorinous and chloro-phenol tastes and odors which are associated with the chlorination of water supplies, and in addition is very efficient bactericidally where adequate contact is available.

2. For the same chlorine dosage the bactericidal velocity of ammonia-chlorine treatment is decidedly less during the first two hours after the treatment is applied than that obtained with chlorine alone. The larger the amount of ammonia present, the greater the lag in bactericidal effect.

3. For periods of contact longer than two hours the bactericidal effect of ammonia-chlorine treatment is greater than that obtained by the use of the same amount of chlorine alone. In this period the ultimate effect is greater with a larger amount of ammonia present.

4. By increasing the ratio of chlorine to ammonia it is possible to obtain a combination which will equal the bactericidal velocity attained after a five minute contact when using chlorine without ammonia. This ratio of chlorine to ammonia, however, did not prevent the formation of chlorophenol tastes and odors in the Chicago water.

5. The temperature of the water has a marked effect on the bactericidal efficiency of the treatment, the efficiency decreasing with the lowering of temperature within the range of the experimental work, 20° to 0°C.

6. An increase in the turbidity of the water naturally decreases the bactericidal efficiency of the ammonia-chlorine process.

With regard to application of ammonia and chlorine, it is best to apply ammonia first and chlorine a short distance in front of ammonia and chlorine should not be too far apart, otherwise, there will be loss of ammonia. Sufficient time should,

however, be allowed before filtration for the proper formation of chloramine, otherwise, filters may reduce ammonia to the extent of 40 to 60 per cent. Ammonia and chlorine must not be mixed together as it may cause explosion. Ammonia can be applied with an ammoniator and chlorine with a chlorinator. Paradon ammoniator and chlorinator are shewn in Figs. 143 and 144 the working of them are described as follows:—

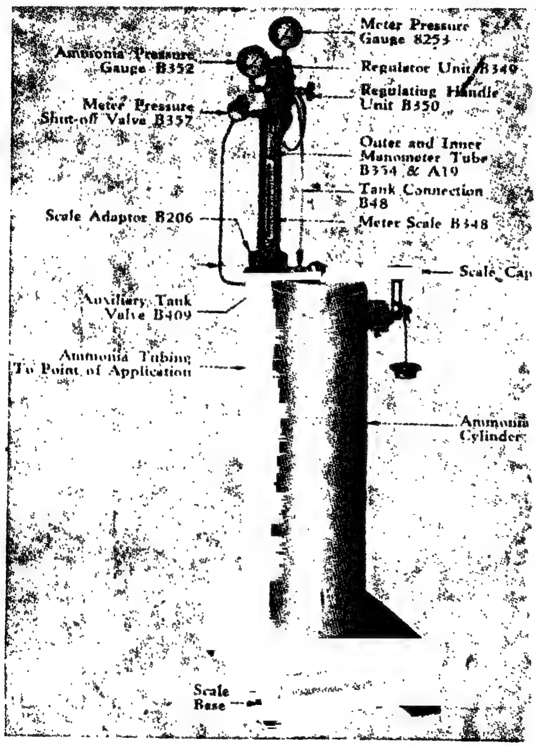


Fig. 143—Paradon Ammoniator.

Ammoniator—The ammonia enters the regulator unit at the gas inlet manifold. It then passes through the inlet valve. The inlet valve reduces the pressure of the ammonia. This inlet valve is actuated by the diaphragm. The ammonia passes along the diaphragm and through the orifice. The flow of the ammonia is regulated by varying the drop in pressure through

the orifice. A manometer indicates the flow of ammonia through the orifice. A bye-pass connection leads from the discharge side of the orifice to the back of the diaphragm of the regulator, through the passage containing the seal screw. This bye-pass connection equalizes the pressure on both sides of the diaphragm

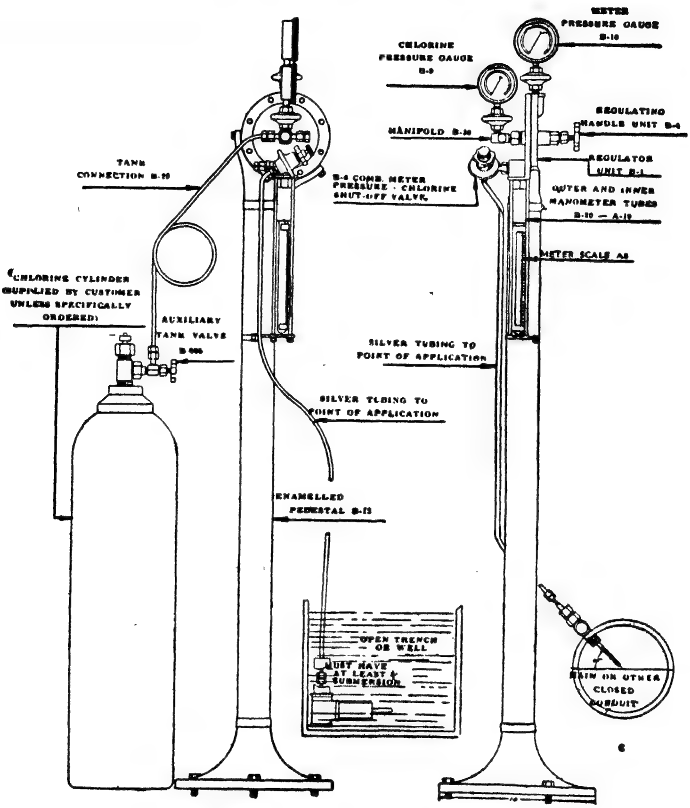


Fig. 144—Paradon Chlorinator.

of the regulator, insofar as changes in ammonia pressure are concerned. The difference in pressure between the front and back of the diaphragm of the regulator and therefore pressure drop through the orifice is, hence, always equal to the compression on the regulator back spring. This compression and, in turn, the flow of ammonia is adjusted by means of the regulating handle unit.

The inner manometer tube connects to the discharge side of the orifice. The bottom of the inner manometer tube is sealed with a liquid. When there is a flow of ammonia through the orifice, the liquid rises in the inner manometer tube and indicates the drop in pressure across the orifice. The height of liquid is converted into terms of ammonia flow by the calibrated meter scale.

After passing through the orifice, the ammonia flows through the meter pressure shut off valve. The function of the meter pressure shut off valve is to maintain the ammonia at a back pressure, indicated by the meter pressure gauge, and equal to the back pressure under which the orifice was empirically calibrated. This is accomplished by adjusting the compression of a spring by means of the adjusting handle. Ammonia will pass through the seat of the meter pressure valve only when the ammonia pressure is sufficient to overcome the compression of the spring. The spring acts directly on the diaphragm of the meter pressure valve. The shut off valve enables the unit to be kept filled with ammonia during periods of shut down, thus preventing access of dirt or moisture.

From the meter pressure shut off valve, the ammonia passes to the point of application.

Chlorinator—The source of supply of chlorine is cylinders of 100 or 150 pounds capacity (chlorine is also available in ton containers), and under a pressure of about 80 pounds per square inch at the usual room temperature of 60°F. The chlorine is in a liquefied state in the chlorine cylinder. The chlorine flows as a gas through the auxiliary tank valve and the flexible tank connection to the regulator unit. The chlorine pressure gauge indicates the pressure of the chlorine in the chlorine cylinder. In the regulator unit the chlorine pressure is reduced and the flow of chlorine measured and controlled.

The chlorine enters the regulator unit at the gas inlet manifold. It then passes through the inlet valve. The inlet valve reduces the pressure of the chlorine. This inlet valve is actuated by the diaphragm. The chlorine passes along the

diaphragm and through the orifice. The flow of the chlorine is regulated by varying the drop in pressure through the orifice. The manometer indicates the flow of chlorine through the orifice. A bye-pass connection leads from the discharge side of the orifice to the back of the diaphragm of the regulator, through the passage containing the seal screw. The difference in pressure between the front and the back of the diaphragm of the regulator and therefore the pressure drop through the orifice is, hence, always equal to the compression on the regulator back spring. This compression and, in turn, the flow of chlorine is adjusted by means of the regulating handle unit.

The inner manometer tube connects to the discharge side of the orifice. The bottom of the inner manometer tube is sealed with a liquid. When there is a flow of chlorine through the orifice, the liquid rises in the manometer tube and indicates the drop in pressure across the orifice. The height of liquid is converted into terms of chlorine flow by the calibrated meter scale.

After passing through the orifice, the chlorine flows through the meter pressure shut-off valve. The function of the meter pressure shut-off valve is to maintain the chlorine at a back pressure, indicated by the meter pressure gauge, and equal to the back pressure under which the orifice was empirically calibrated. This is accomplished by adjusting the compression of the spring by means of the adjusting handle. Chlorine will pass through the seat of the meter pressure shut-off valve only when the chlorine pressure is sufficient to overcome the compression of the spring. The spring acts directly on the diaphragm of the meter pressure shut-off valve, and to which the needle and holder are attached.

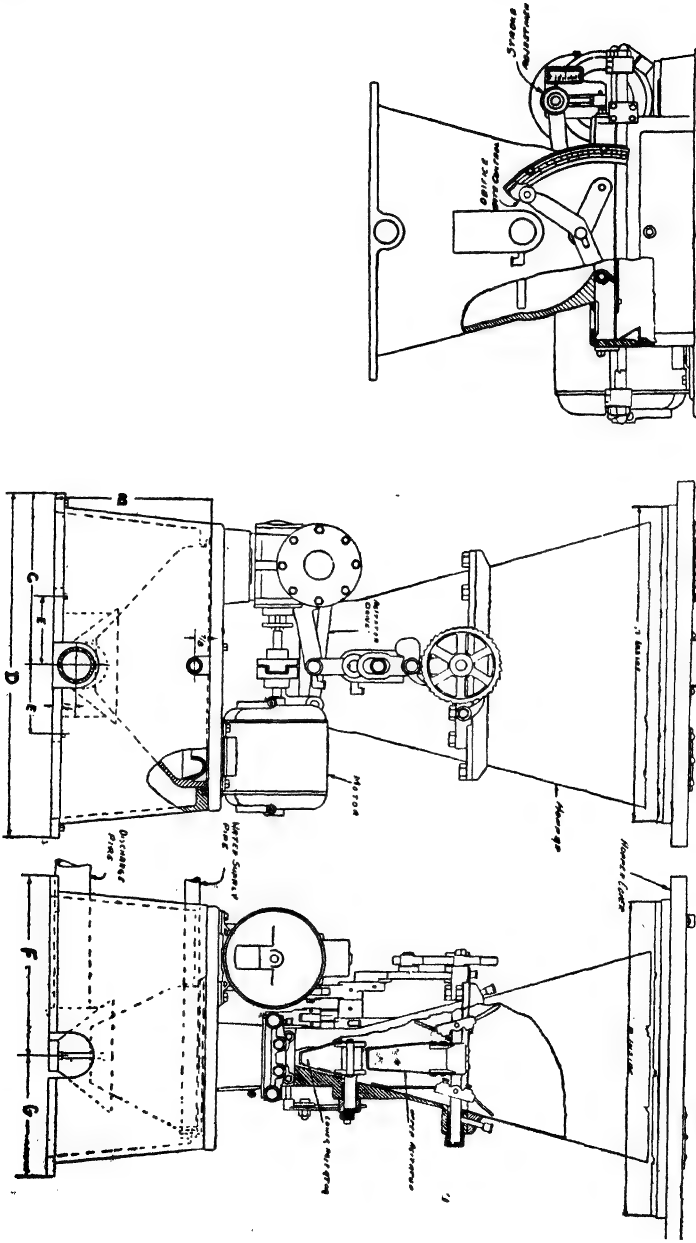
From the meter pressure shut-off valve, the chlorine passes to the diffuser at the point of application.

Activated Carbon Treatment—The use of carbon as an auxiliary to filtration is not new in Bengal, some of the sand filters constructed by Mr. A. E. Silk, Sanitary Engineer, Bengal, about the year 1899 had a layer of charcoal beds below

the layer of fine sand. Charcoal beds for filtration were also in use in many places in Europe several years ago. With regard to the use of activated carbon in connection with water purification, the credit of a pioneer is perhaps due to the Candy Filters & Company of England. During last ten years extensive research has, however, been made both in America and Germany as to the possibilities of activated carbon in the field of water purification, and now activated carbon is universally recognised as a potent material to combat objectionable taste and odour.

Activated carbon is generally obtained by activating primary carbon by heating it to a high temperature and by passing steam or small quantity of air through it in closed retorts. The primary carbon may be obtained by charring under low temperature carbonaceous materials like wood, peat and lignaite etc. Activated Carbon is a valuable adsorbent and by virtue of this property it removes almost any kind of taste and odour. According to many authorities, there is no taste producing compound likely to be present in water which cannot be adsorbed by this method.

Writing about the activation of carbon Baylis writes, "If Chaney's theory of adsorption of hydrocarbons on the surface of active carbon in carbonization of carbonaceous materials at low temperature is correct, then the activation consists in removing these adsorbed hydrocarbons from the primary carbon. When the carbon is heated to a very high temperature so as to decompose the adsorption complex, that is, to cause the hydrocarbons to be released from the primary carbon, it is released at a temperature much above the normal decomposition point of the hydrocarbons. The hydrocarbons then may break down and form inactive carbon that deposits on the surface of the active carbon. It has been stated that the formation of carbon below a certain temperature produces at least a part of the carbon in the active state or capable of being activated, and when formed above the critical temperature the carbon is largely inactive and cannot be made active. It is thus seen



Front View.

End View.

Fig. 145—Dry Chemical Feede

Rear View.

that the mere heating of a carbonized material will not produce carbon of high adsorptive capacity.

The hydrocarbon constituents of the adsoption complex appear to be slightly less resistant to oxidation than the active carbon, and by a proper adjustment of the temperature and concentration of the oxidizing material the adsorbed hydrocarbons may be removed. This requires some care, but when properly done very little inactive carbon is produced to cover the surface of the active material. The oxidizing agent used to the greatest extent are air and steam. Carbon dioxide and chlorine have been used to a limited extent.

Barker believes that the mechanism of charcoal activation partly consists of converting the primary charcoal to graphite or graphite-like carbon of crystalline structure, and partly of increasing the interior surface area by producing a large volume of very small capillaries. The hydrogen and oxygen content of the charcoal is markedly decreased by activation and the true density of the carbon material increased from about 1·45 to 2·15."

Formerly activated carbon in water purification was used in the form filter beds, but with the introduction of carbon in the powdered form it is fed into water like other chemical used in water purification by means of dry feed machine (Fig. 145) and water eductor. Activated carbon is of such low density and so finely powdered that it settles very slowly when mixed with water. When applied to water before filtration most of it is deposited on the surface of the filter bed forming a film functioning as an adsorbing filter bed. This coating on the surface of filters can easily be removed by usual back washing. The most efficient point of application, however, can only be determined by experiment. Activated carbon requires to be intimately mixed with water in mixing basins like other chemicals. Rates of application of the activated carbon varies from 0·1 to 1·0 part per 100,000 of settled water depending upon the intensity of taste or odor to be removed, but to utilise it, however, to the best advantage the dose should be worked out specifically at each plant as it depend considerably upon the local conditions.

APPENDIX XI

SPECIFICATION FOR MATERIALS USED IN WATER PURIFICATION.

(Extract from American Water Works Practice).

Sulphate of Ammonia.—This material shall contain not less than 17 per cent of available water soluble Alumina (Al_2O_3) and of this there must be at least 2·5 per cent of its weight in excess of the quantity required to combine with Sulphuric Acid present. The material shall contain not more than 7·5 per cent of matter insoluble in cold distilled water.

Soda Ash.—The soda ash shall be that known as 58 per cent soda ash and shall contain not less than 98 per cent of pure Sodium Carbonate. It shall be in dry powdered form and free from foreign matter.

Caustic Soda.—The material shall be that known as 76 per cent actual best Sodium Oxide and shall contain not less than 98 per cent of Sodium Hydroxide ($NaOH$).

Lime.—Quick lime shall contain 85 per cent of Calcium Oxide (CaO) and hydrated lime—95 per cent of Calcium Hydroxide $Ca(OH)_2$. Quick lime when immersed in water shall readily disintegrate in a suspension of finely devided material.

Bleaching Powder. The material shall contain not less than 33 per cent. of available Chlorine.

Sulphate of Iron.—This material shall contain not less than 98 per cent of pure Ferrous Sulphate and not more than 1/15 per cent of free acid and shall be clean and free from all dirt and particles of foreign matters.

Activated Carbon.—Baylis suggests:—"Activated carbon is a carbonized material in which part of the surface atoms of the carbon will adsorb certain other compounds. It is formed when carbonaceous substances are first carbonized in closed vessels and then given special heat treatment so as to leave the

surface atoms in a condition in which they will adsorb such substances as to tend to unite with carbon. Ordinary charcoal has very little adsorption capacity and is not classified with active carbons.

The carbon is to be applied into the water in the powdered form and it must be of such fineness that it will not settle rapidly. At least 75 per cent of the material shall pass a 200—mesh sieve and at least 95 per cent shall pass a 100-mesh sieve.

Thirty-five parts per million of the carbon should reduce the phenol in a solution which contains 0·10 parts per million of phenol to 0·01 part per million when stirred for two hours. A material in which 35 parts per million added to the water just produces this reduction is regarded as having the specified adsorption capacity.

APPENDIX XII.

ANNUITY TABLE.

Repayment of A Loan by Way of Annuity.

Annual Payment to include Principal and Interest.

For Re. 1/-

TABLE I.

Rate of interest.	1 year.	3 years.	5 years.	7 years.	9 years.	10 years.	12 years.	14 years.	15 years.	17 years.	20 years.	23 years.	25 years.	28 years.	30 years.
2 p.c.	1·02	·34675	·21215	·15451	·12251	·11132	·09455	·08260	·07782	·06996	·06115	·05466	·05122	·04698	·04464
2¼ ,,	1·0025	·34844	·21370	·15600	·12398	·11278	·09601	·08406	·07928	·07144	·06264	·05617	·05273	·04852	·04619
2½ ,,	1·025	·35013	·21524	·15749	·12545	·11425	·09748	08553	·08076	·07292	·06414	·05769	·05427	·05008	·04777
2¾ ,,	1·0275	·35183	·21679	·15899	·12694	·11573	·09896	·08702	·08225	·07443	·06567	·05924	·05583	·05167	·04938
3 ,,	1·03	·35353	·21835	·16050	·12843	·11723	·10146	·08852	·08376	·07595	·06721	·06081	·05742	·05329	·05101
3¼ ,,	1·0325	·35523	·21991	·16202	·12993	11873	·10196	·09004	·08528	·07748	·06877	·06240	·05903	·05493	·05268
3½ ,,	1·035	·35693	·22148	·16354	·13144	·12024	·10348	·09157	·08682	·07904	·07036	·06401	·06067	·05660	·05437
3¾ ,,	1·0375	·35864	·22305	·16507	·13296	·12176	·10501	·09311	·08837	·08061	·07196	·06565	·06233	05829	·05608
4 ,,	1·04	·36034	·22462	·16660	·13449	·12329	·10655	·09466	·08994	·08219	·07358	·06730	·06401	·06001	·05783
4¼ ,,	1·0425	·36205	·22620	·16815	·13602	·12483	·10810	·09623	·09152	08380	·07521	·06898	·06571	·06175	·05959
4½ ,,	1·045	·36377	·22779	·16970	·13757	·12637	·10966	·09782	·09311	·08541	·07687	·07068	·06743	·06352	·06139
4¾ ,,	1·0475	·36548	·22938	·17125	·13912	·12793	·11124	·09941	·09472	·08705	·07855	·07239	·06918	·06531	·06320
5 ,,	1·05	·36720	·23097	·17281	·14069	·12950	·11282	·10102	·09634	·08369	·08024	·07413	·07095	·06712	·06505
5¼ ,,	1·0525	·36893	·23257	·17438	·14226	·13108	·11444	·10264	09797	·09036	·08195	·07589	·07274	·06895	·06691
5½ ,,	1·055	·37065	·23417	·17596	·14383	·13266	·11602	·10427	·09962	·09204	·08367	·07766	·07454	·07081	·06880
5¾ ,,	1·0575	·37238	·23578	·17754	·14542	·13426	·11764	·10592	·10128	·09373	·08542	·07946	·07637	·07269	·07071
6 ,,	1·06	·37410	·23739	·17913	·14702	·13586	·11927	·10758	·10296	·09544	·08718	·08127	·07822	·07459	·07264
6¼ ,,	1·0625	·37584	·23901	·18072	·14862	·13748	·12091	·10925	·10465	·09716	·08896	·08311	·08009	·07651	·07460
6½ ,,	1·065	·37757	·24063	·18233	·15023	·13910	·12256	·11094	·10635	·09890	·09075	·08496	·08198	·07845	·07657
6¾ ,,	1·0675	·37931	·24226	·18393	·15185	·14073	·12422	·11263	·10806	·10065	·09256	·08682	·08388	·08041	·07857
7 ,,	1·07	·38105	·24389	·18555	·15348	·14237	·12590	·11434	·10979	·10242	·09439	·08871	·08581	·08239	·08058

DEPRECIATION TABLE.

$$D = \frac{r}{(1+r)^l - 1}$$

D = depriciation per cent.

r = interest per cent.

l = life rate of interest.

TABLE II.

Life	3%	3½%	4%	4½%	5%	6%	7%
5	18·84	18·65	18·47	18·28	18·10	17·74	17·39
10	8·72	8·52	8·33	8·14	7·95	7·59	7·24
15	5·38	5·18	5·00	4·81	4·63	4·30	3·98
20	3·72	3·54	3 36	3·19	3·02	2·72	2·44
25	2·74	2·57	2·40	2·24	2·10	1·82	1·58
30	2·10	1·94	1·78	1·64	1·51	1·26	1·06
35	1·65	1·50	1·36	1·23	1·11	·90	·72
40	1·33	1·18	1·05	·93	·83	·64	·50
45	1·08	·95	·83	·72	·63	·47	·35
50	·89	76	·66	·56	·48	·35	·25
60	·61	·51	·42	·35	·28	·19	·12
70	·43	·35	·27	·22	·17	·10	·06
75	·31	·29	·22	·17	·13	·08	·04
80	·31	·24	·18	·14	·10	·06	·03
90	·23	·17	·12	·09	·06	·03	·02
100	·16	·12	·08	·06	·04	·02	·008

Example 1. If a loan of Rs. 1,00,000 be borrowed to be repaid in equal annual instalment in 30 years at an interest of 6 per cent. What is the annual instalment?

The annual instalment from the table will appear to be Rs. 0·07264 × 1,00,000 Rs. 7,264/-

Example 2. If a loan of Rs. 50,000 be borrowed at 6 per cent. rate of interest to be repaid in half-yearly equal instalment in 14 years. What will be the half-yearly instalment?

The half-yearly instalment will be equal to a loan repayable in 14 × 2 = 28 years at a rate of interest of 6/2 i.e., 3 per cent. or equal to Rs. 0·05329 × 50,000 = Rs. 2,664·5.

Example 3. To find the depriciation of a plant consisting of centrifugal pump driven by an oil engine. The cost of centrifugal pump is taken to be Rs. 7,000/- and that of oil engine to be Rs. 32,000/- and the rate of interest is 4 per cent. The life of centrifugal pump is assumed to be 20 years and that of oil engine to be 30 years.

The depreciation charge will appear from the depreciation table to be

For the pump ...	Rs. 7,000 × 3·36%	Rs. 235·2
For the engine ...	Rs. 32,000 × 1·78%	Rs. 569·6
Annual Depreciation charge ...	...	Rs. 804·8

APPENDIX XIII.

TABLE I.

HYDRAULIC MEMORANDA.

1 Cubic inch of water ...	0·03616 lbs.; 0·003607 gallon.
1 Cubic foot of water ...	62·484 lbs. 0·5579 cwt. 0·0279 ton. 6·2484 gallons.
36 Cubic feet of water ...	1 ton; 2,240 lbs.
1 Cwt. of water	1·7925 cubic feet. 11·20 gallons.
1 Ton of water	35·85 cubic feet. 224·0 gallons.
1 lb. of water	27·7463 cubic inches. 0·1606 cubic foot. 0·10 gallon.
1 Gallon of water	277·24 cubic inches. 0·16 cubic foot. 10·0 lbs.
Water in freezing expands to extent of	8½ per cent.
1 lb. per sq. in pressure ...	2·31 feet head of water (approx.).
1 Atmosphere	34·0 feet head head of water (approx.).
1 Inch of rain fall over an acre	101·00 tons. 22,622·5 gallons.
1 Cubic foot of water per second	373·6 gallons per minute. 22,420 gallons per hour. 538,080 gallons per day.
A column of water 1 foot high exerts pressure	0·433 lbs. per sq. inch. 62·352 lbs. per sq. foot.
1 Inch rain fall per hour ...	0·5 cusec per acre.

1 Inch rain yields about ½ gallon of water per sq. foot.

1 Inch rain in a year per acre yields 62 gallons per day.

1 Inch rain in a year per square mile yields (if stored) about 38,000 gallons per day.

Inches of rain fall × 14·48 ...	millions of gallons per square mile.
Cubic inches × 0·003616 ...	gallons.
Cubic feet × 6·2484	gallons.
Cubic inches × 227·463 ...	gallons.
Gallons × 0·1601	cubic feet.
Cylindrical feet × 4·9075 ...	gallons.

Degrees centigrade to degrees fahrenheit; multiply by 9, divide by 5, and add 32.

Parts per 100,000 into grains per gallon, multiply by 7, and divide by 10.

Grains per gallon into parts per 100,000 multiply by 10 and divide by 7.

Grams per litre into grains per gallon, multiply by 70.

Grams per litre into parts per 100,000 multiply by 100.

Table II.

TABLE OF EFFICEINCIES.

*BOILERS.

	Efficiency, Per Cent.
Ordinary boilers with hand finring and bituminous coal ...	70 to 75
Ditto. with anthracite coal & hand firing	73 to 78
Ordinary boilers with economiser	80 to 84
Large boilers with mechanical strokers and bituminous coal	78 to 80
Large boilers with oil fuel	82
Large boilers with feed water heater from waste gases, economiser & mechanical stokers	88 to 90
Producer	65 to 80

STEAM ENGINES.

	Mechanical Efficienty.
Steam engines	80 to 90
Steam turbine (Small)	60 to 70
Steam turbine (Large)	80 to 85

OIL ENGINES.

Semi-Diesel engines	74 to 80
Diesel engines (Constant pressure Cycle)	78 to 80
Diesel Engines (High compression, solid injection) ...	80 to 84

GAS ENGINES.

Gas engines (50 to 500 B. H. P.)	75 to 80

* In practice about 10 to 25 per cent. below thc above efficiencies is usually obtained.

ELECTRIC PLANTS.

	Mechanical Efficienty.
Alternating Current Generator	90 to 95
Direct Current Generator	85 to 93
Alternating Current Motors	80 to 91
Direct Current Motors	82 to 93
Transformers	95 to 98

GEARS.

Helical gears	90 to 95
Worm gears	90 to 95
Spur gears	90 to 98
Bevel gear	85 to 90
Frictional clutch	99 to 100
Belting	97 to 98

PUMPS.

Piston Pumps :—	
Small duplex, direct acting	40 to 60
Compound, direct acting, non-condensing	70 to 90
Ditto with "high duty" attachment	60 to 80
Single-cylinder, flywheel, non-condensing	70 to 90
Multi-cylinder, flywheel, non-condensing	70 to 90
Multi-cylinder, flywheel, condensing	70 to 95
Geared pumps	50 to 90
Centrifugal pumps :—	
Single-stage for low-heads	55 to 75
Multi-stage for high-heads	40 to 60
Rotary Pumps	50 to 80
Air lift pumps	20 to 35
Turbo-pumps	60 to 70

FILTERS.

	Bacterial Efficiency.
Slow Sand Filters	99·9
Rapid Sand or Mechnical Filters	90—99

TABLE III.

TABLE OF STEAM & FUEL CONSUMPTION OF PRIME-MOVERS.

	lbs. of steam per I.H.P. hour.
Simple non-condensing engines	20 to 40
Simple condensing engines, with steam at 60 lbs. pressure, and fitted with expansion gear	19 to 22
Compound condensing engines, with steam at 60 lbs. pressure	18 to 20
Compound condensing engines, with steam at 100 lbs. pressure	16½ to 18½
Triple expansion condensing engines with steam at 160 lbs. pressure	13 to 16
	lbs. per B.H.P. hour.
Turbines	7·5 to 18
	lbs. of steam per H.P. hour of useful work.
Pulsometer	100 to 400
	B.Th.U's per B.H.P. hour.
Gas engines	10,000 to 12,000
Oil engines :—	
	lbs. per B.H.P. hour.
Semi-Diesel	0·41 to 0·46
	lbs. per B.H.P. hour.
Diesel	0·38 to 0·41

CAPACITY OF DIFFERENT KINDS OF FILLERS

TABLE V.

Area of Filter in sq. feet.	Slow Sand Filters.		Mechanical Filters.			
			Gravity Type.		Pressure Type.	
	Per hour.	Per day.	Per hour.	Per day.	Per hour.	Per day.
	gallons.	gallons.	gallons.	gallons.	gallons.	gallons.
1	1·5	36·0	70·0	1,680	100	2,400
10	15	360	700	16,800	1,000	24,000
20	30	720	1,400	33,600	2,000	48,000
30	45	1,080	2,100	40,400	3,000	72,000
40	60	1,440	2,800	67,200	4,000	96,000
50	75	1,800	3,500	84,000	5,000	1,20,000
60	90	2,160	4,200	90,800	6,000	1,44,000
70	105	2,520	4.900	97,600	7,000	1,68,000
80	120	2,880	5,600	1,34,400	8,000	1,92,000
90	135	3,240	6,300	1,51,200	9,000	2,16,000
100	150	3,600	7,000	1,68,000	10,000	2,40,000
200	300	7,200	14,000	3,36,000	20,000	4,80,000
300	450	10,800	21,000	4,04,000	30,000	7,20,000
400	600	14,400	28,000	6,72,000	40,000	9,60,000
500	750	18,000	35,000	8,40,000	50,000	12,00,000

RELATION BETWEEN RATE OF FILTRATION AND VERTICAL VELOCITY.

TABLE VI.

Gallons Per Sq. Ft. Per. Hour.	Gallons Per Sq. Ft. in 24 Hours.	Cubic Feet in 24 Hours.	Vertical Velocity in Inches Per Hour.
1	24	3·85	1·93
2	48	7·71	3·85
3	72	11·56	5·78
4	96	15·42	7·71
5	120	19·27	9·64
6	144	23·12	11·56
7	168	26·98	13·49
8	192	30·83	15·42
9	216	34·69	17·45
10	240	38·54	19·27
20	480	77·09	38·54
30	720	115·63	57·82
40	960	154.·17	77·09
50	1,200	192 72	96·36
60	1,440	231·26	115·63
70	1,680	269·80	134·90
80	1,920	308·35	154·17
90	2,160	346·89	173·45
100	2,400	385·44	192·72

CONTENTS OF CIRCULAR TANKS IN GALLONS

TABLE VII.

Dia. in ft.	Depth in feet.									
	1	2	3	4	5	6	7	8	9	10
5	123	246	369	492	615	728	861	984	1,107	1,230
6	177	354	531	708	885	1,062	1,239	1,416	1,593	1,770
7	240	480	720	960	1,200	1,440	1,680	1,920	2,160	2,400
8	314	628	942	1,256	1,570	1,884	2,198	2,512	2,826	3,140
9	397	794	1,191	1,588	1,985	2,382	2,779	3,176	3,573	3,970
10	491	982	1,473	1,964	2,455	2,946	3,437	3,928	4,419	4,910
11	594	1,188	1,782	2,376	2,970	3,564	4,158	4,752	5,346	5,940
12	707	1,414	2,121	2,828	3,535	4,242	4,949	5,656	6,363	7,070
13	829	1,658	2,487	3,316	4,145	4,974	5,803	6,732	7,461	8,290
14	962	1,924	2,886	3,848	4,810	5,772	6,734	7,696	8,658	9,620
15	1,104	2,208	3,312	4,416	5,520	6,624	7,728	8,832	9,936	11,040
16	1,257	2,514	3,771	5,028	6,285	7,542	8,799	10,056	11,313	12,570
17	1,419	2,838	4,257	5,676	7,095	8,514	9,933	11,352	12,771	14,190
18	1,591	3,182	4,773	6,364	7,955	9,546	11,137	12,728	14,319	15,910
19	1,772	3,544	5,316	7,088	8,860	10,632	12,404	14,176	15,948	17,720
20	1,964	3,928	5,892	7,856	9,820	11,784	13,748	15,712	17,676	19,640
21	2,165	4,330	6,495	8,660	10,825	12,990	15,155	17,320	19,485	21,650
22	2,376	4,752	7,128	9,504	11,880	14,256	16,632	19,008	21,384	23,760
23	2,597	5,194	7,791	10,388	12,985	15,572	18,179	20,776	23,373	25,970
24	2,828	5,656	8,484	11,312	14,140	16,968	19,796	22,624	25,452	28,280
25	3,068	6,136	9,204	12,272	15,340	18,408	21,476	24,544	27,612	30,680
26	3,318	6,636	9,954	13,272	16,590	19,908	23,226	26,544	29,862	33,180
27	3,579	7,158	10,737	14,316	17,895	21,474	25,053	28,632	32,211	35,790
28	3,849	7,698	11,547	15,396	19,245	23,094	26,943	30,792	34,641	38,490
29	4,128	8,256	12,384	16,512	20,540	24,768	28,896	33,024	37,152	41,280
30	4,418	8,836	13,254	17,672	22,090	26,508	30,926	35,344	39,762	44,180
35	6,013	12,026	18,039	24,052	30,065	36,078	42,091	48,104	54,117	60,130
40	7,857	15,714	23,571	31,428	39,285	47,142	54,999	62,856	70,713	78,570
45	9,938	19,876	29,814	39,752	49,690	59,628	69,566	79,504	89,442	99,380
50	12,268	24,536	36,804	49,072	61,340	73,608	85,876	98,144	1,10,412	1,22,680
60	17,669	35,338	53,007	70,676	88,345	1,06,014	1,23,683	1,41,352	1,59,021	1,76,690
70	24,050	48,100	72,150	96,200	1,20,250	1,44,300	1,68,350	1,92,400	2,16,450	2,40,500
75	27,613	55,226	82,839	1,10,452	1,38,065	1,65,678	1,93,291	2,20,904	2,48,517	2,76,130

TABLE OF WATER-HEADS, EQUIVALENT PRESSURES, WORK AND HORSE POWER

TABLE VIII.

Head in feet.	Equivalent Pressure in pounds per square inch.	Foot-pounds of work when raising 1,000 gallons per minute against corresponding heads.	Corresponding Water Horse-Power.	Head in feet.	Equivalent Pressure in pounds per square inch.	Foot-pounds of work when raising 1,000 gallons per minute against corresponding heads.	Corresponding Water Horse-Power.
1	0·43	1,000	0·03	80	34·64	80,000	2·42
2	0·87	2,000	0·06	90	38·97	90,000	2·73
3	1·30	3,000	0·09	100	43·30	100,000	3·03
4	1·73	4,000	0·12	200	86·60	200,000	6·06
5	2·17	5,000	0·15	300	129·90	300,000	9·09
6	2·60	6,000	0·18	400	173·20	400,000	12·12
7	3·30	7,000	0·21	500	216·50	500,000	15·15
8	3·46	8,000	0·24	600	259·80	600,000	18·18
9	3·90	9,000	0·27	700	303·10	700,000	21·21
10	4·33	10,000	0·30	800	346·40	800,000	24·24
20	8·66	20,000	0·61	900	389·70	900,000	27·27
30	12·99	30,000	0·91	1,000	433·00	1,000,000	30·30
40	17·32	40,000	1·21	1,250	541·25	1,250,000	37·88
50	21·65	50,000	1·52	1,500	649·50	1,500,000	45·45
60	25·98	60,000	1·82	1,750	757·75	1,750,000	53·04
70	30·31	70,000	2·12	2,000	866·00	2,000,000	60·60

CONTENTS AND WEIGHTS OF WATER IN CYLINDER

TABLE IX.

Diameter of Pipes in inches.	Equivalents of diameter of Pipes in feet.	Area of Cross Section of Pipe in square feet.	Contents per foot run in cubic feet.	Contents per foot run in gallons.	Weight of contents per foot run in lbs.	Diameter of Pipes in inches.	Equivalents of diameter of Pipes in feet.	Area of Cross Section of Pipe in square feet.	Contents per foot run in cubic feet.	Contents per foot run in gallons.	Weight of contents per foot run in lbs.
1	0·083	0·0055	0·0055	0·034	0·34	13	1·083	0·9220	0·9220	5·751	58
2	0·167	0·0220	0·0220	0·136	1	14	1·167	1·0690	1·0690	6·671	67
3	0·250	0·0490	0·4090	0·306	3	15	1·250	1·2270	1·2270	7·656	77
4	0·333	0·0870	0·0870	0·545	5	16	1·333	1·3960	1·3960	8·712	87
5	0·417	0·1360	0·1360	0·851	8	17	1·417	1·5760	1·5760	9·835	98
6	0·500	0·1960	0·1960	1·226	12	18	1·500	1·7670	1·7670	11·030	110
7	0·583	0·2670	0·2670	1·668	17	19	1·583	1·9690	1·9690	12·280	123
8	0·667	0·3490	0·3490	2·411	24	20	1·667	2·1820	2·1820	13·610	136
9	0·750	0·4220	0·4220	2·757	28	21	1·750	2·4050	2·4050	15·010	150
10	0·833	0·5450	0·5450	3·403	34	22	1·833	2·6400	2·6400	16·470	165
11	0·917	0·6600	0·6600	4·118	41	23	1·917	2·8850	2·8850	17·80	178
12	1·000	0·7850	0·7850	4·901	49	24	2·000	3·1420	3·1420	19·61	196

CONVERSION TABLE.

TABLE X.

	United States Gallon.	Imperial Gallon.	Litres.	Cubic Foot.	Pounds Avoirdupois.	Grains.	Grams.
1 United States Gallon ..	1·000	0·831	3·79	0·134	8·34	58,418·0	3,785·0
1 Imperial Gallon	1·200	1·000	4·54	0·160	10·00	70,118·0	4,544·0
1 Litre	0·264	0·220	1·00	0·035	2·20	15,432·0	1,000·0
1 Cubic Foot	7·480	6·230	28·32	1·000	62·38	436,996·0	28,317·0
1 Pound, avoirdupois ...	0·120	0·144	0·454	0·016	1·00	7,000·0	453·6
1 Grain	0·000017	0·00002	0·000065	0·0000023	0·0001	1·0	0·065
1 Gram	0·00026	0·00031	0·001	0·000035	0·0022	15·43	1·000

READY RECKONER OF DOSAGE OF CHEMICALS.

TABLE XI.

Grains per Gallon.	Parts per 100,000.	Pounds per 100,000. Gallons.	Grains per Gallon.	Parts per 100,000	Pounds per 100,000 Gallons.	Parts per 100,000.	Grains per Gallon.	Pounds per 100,000. Gallons.	Parts per 100,000.	Grains per Gallon.	Pounds per 100,000 Gallons.
0·1	0·1429	1·429	0·6	0·8571	8·571	0·1	0·07	1·00	0·6	0·42	6·00
0·2	0·2857	2·857	0·7	1·0000	10·000	0·2	0·14	2·00	0·7	0·49	7·00
										0·56	
0·3	0·4286	4·286	0·8	1·1429	11·429	0·3	0·21	3·00	0·8		8·00
0·4	0·5714	5·714	0·9	1·2857	12·857	0·4	0·28	4·00	0·9	0·63	9·00
0·5	0·7143	7·143	1·0	1·4286	14·286	0·5	0·35	5·00	1·0	0·70	10·00

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